Field and Laboratory Studies on High-Mast Lighting Towers in Iowa

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ABSTRACT

Recent failures of high-mast lighting towers in several Midwestern states have raised questions as to the robustness and safety of the existing inventory of similar structures. Failure of these structures is very critical, as they are typically located adjacent to interstates or other high-speed highways. The potential exists for these fracture-critical structures to fall across multiple traffic lanes or adjacent property. Forensic studies on cracked poles in the state of Iowa and elsewhere have revealed fatigue to be the primary cause of the failures.

Interestingly, there is little data on the actual response of these structures in terms of natural wind, long-term cyclic stresses, and general dynamic properties. Believed to be the first of its kind, the Iowa Department of Transportation has sponsored a comprehensive field testing and long-term remote monitoring program that has focused on two poles in near Clear Lake, Iowa. The results of the field studies suggest that additional fatigue cracking can be expected at other poles in the future. In order to address this issue, retrofit strategies have been developed and are currently going through static and fatigue tests to establish the fatigue resistance and long-term viability of the design. The laboratory tests, which have just begun, have been augmented with detailed finite element analytical studies on the base plate connection. This paper will report on the initial results of the field studies and provide suggested strategies for retrofit, inspection, and maintenance of these fracture-critical structures.

Key words: fatigue—field testing—high-mast tower

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BACKGROUND

Following the November 12, 2003, collapse of a 140 ft. high-mast lighting tower along I-29 in Sioux City, Iowa, an intensive investigation into the cause of failure was carried out (Dexter 2004). Subsequently, two high-mast towers near Clear Lake, Iowa, were field-tested and monitored for one year, from October 2004 through November 2005. One of the towers was retrofitted with a steel reinforcing jacket at its base, while the other remained as originally designed (Connor and Hodgson 2006).

An additional phase of testing has been undertaken to further study the behavior of the reinforcing jacket and is the subject of this preliminary report. The as-built tower located near Clear Lake, Iowa, was retrofitted in June 2006 with a modified reinforcing jacket (designed and installed by Wiss, Janney, Elstner, and Associates). Static and dynamic load tests were performed, and long-term monitoring of the tower is currently underway. This report highlights the preliminary findings of this current phase of work.

Objectives of the Current Study

The current study was initiated to quantify the stresses induced in critical details on the reinforcing jacket and the tower itself through the use of field instrumentation, testing, and long-term monitoring.

Strain gages were installed on the both the tower and the reinforcing jacket. Additional strain gages were installed on two anchor rods. Tests were conducted with and without the reinforcing jacket installed. Data were collected from all strain gages during static load testing and were used to study the stress distribution of the tower caused by known loads, both with and without the reinforcing jacket. The tower was tested dynamically by first applying a static load and then quickly releasing the load, causing the tower to vibrate freely. Dynamic properties of the tower, such as modal frequencies and damping ratios, are obtained by reducing the data.

Summary of the Field Testing Program

Installation of all sensors and load testing were conducted during the week of June 26, 2006. The tower is located at the I-35/US 18 interchange near Clear Lake, Iowa. It is denoted tower number 1 of the interchange. (This tower was termed the “as-built tower” in reference [Connor and Hodgson 2006].)

A series of static and dynamic loading tests were conducted. These tests were conducted by statically loading the towers with a cable fixed at one end and connected to the tower approximately 35 ft. above the base. The load was subsequently released rapidly to allow the tower to vibrate freely. These dynamic, or “pluck,” tests produced a free decay vibration signature that is used to extract both the natural frequencies and damping characteristics of the high-mast tower.

In addition to the load testing, a 12-month long-term monitoring program is currently underway to quantify the response of the tower under natural wind loading. During the long-term monitoring period, ambient vibration data are recorded (for 15 to 30 minutes) when wind speeds and/or tower stresses exceed predetermined trigger levels. Additionally, stress-range histograms are continuously developed. Furthermore, wind speed and direction are continuously monitored.

INSTRUMENTATION PLAN AND DATA ACQUISITION

The following sections describe the sensors and instrumentation plan used during the static/dynamic testing and the long-term monitoring programs.

Strain Gages

Strain gages were placed at predetermined locations. All strain gages installed in the field were produced by Measurements Group, Inc. and were 0.25 in. gage length, model LWK-06-W250B-350. These gages are uniaxial weldable resistance-type strain gages. Weldable-type strain gages were selected due to the ease of installation in a variety of weather conditions. The welds are point- or spot-resistance welds about the size of a pin prick. The probe is powered by a battery and only touches the foil that the strain gage is mounted on by the manufacturer. This fuses the foil to the steel surface. It takes 40 or more of these small welds to attach the gage to the steel surface. There are no arc strikes or heat-affected zones that are discernible. There is no preheat or any other preparation involved other than the preparation of the local metal surface by grinding and then cleaning before the gage is attached to the component with the welding unit. There has never been an instance of adverse behavior associated with the use of weldable strain gages, including their installation on extremely brittle material such as A615 Gr75 steel reinforcing bars.

These strain gages are also temperature compensated and perform very well when accurate strain measurements are required over long periods of time (months to years). The gage resistance is 350 ohms, and an excitation voltage of 10 volts was used. All gages were protected with a multilayer weatherproofing system and then sealed with a silicon-type compound.

Accelerometers

Two uniaxial accelerometers were used for the dynamic tests only. The accelerometers were manufactured by PCB Piezotronics, Inc. (model 3701G3FA3G). This accelerometer has a peak measurable acceleration of 3 g.

These accelerometers are termed capacitive (or DC) accelerometers. The primary component of these sensors is an internal capacitor. When subjected to acceleration, the sensor outputs a voltage in direct proportion to the magnitude of the acceleration. They are specifically designed for measuring low-level, low-frequency accelerations, such as those found on a bridge or a high-mast lighting tower.

Anemometer

An anemometer is used to measure wind speed and direction and is installed atop a 30 ft. wooden telephone pole directly adjacent to the high-mast tower. The anemometer (model number 5103) is manufactured by R.M. Young, Inc. and is a propeller-type anemometer. Both wind speed and wind direction are measured.

Data Acquisition System

A Campbell Scientific CR9000 data logger was used for the collection of data during all static and dynamic testing and continues to be used for the long-term monitoring phase. This logger is a high-speed, multichannel 16-bit data acquisition system. The data logger was configured with digital and analog filters to assure noise-free signals.

The data logger is enclosed in a weather-tight box adjacent to the tower. Remote communications with the data logger are maintained through a satellite internet connection. Data collection is performed automatically. The satellite link is also used to upload new programs as needed. Data are collected and reviewed periodically to assure the integrity of the data.

Instrumentation Plan

A total of 25 strain gages were applied to the tower. Of these, 12 were installed on the existing tower, 9 were installed on the reinforcing jacket, and the remaining 4 gages were installed on two anchor rods. Key drawings are presented in Figure 1.

On the existing tower, a set of four strain gages was installed 90 degrees apart at a section 6 ft. above the base plate. Two strain gages were placed on the tower 180 degrees apart approximately 53 in. above the baseplate (just below the top of the reinforcing jacket). Two strain gages were placed on the tower 180 degrees apart 12 in. above the baseplate. Finally, a set of four strain gages was installed 90 degrees apart adjacent to the base weld. All of these strain gages were oriented vertically. The inside surface of the reinforcing jacket was ground out at strain gaged locations so pressure would not be applied to the strain gage after the bolted jacket connection was fully tightened.

On the reinforcing jacket, two strain gages were installed 180 degrees apart approximately 53 in. above the baseplate (just below the top of the reinforcing jacket). Two strain gages were installed 180 degrees apart approximately 21 in. above the baseplate. Finally, five strain gages were placed around the perimeter of the reinforcing jacket adjacent to the base weld. Again, all of these strain gages were oriented vertically.

Two anchor rods on the north side of the tower were instrumented. On each anchor rod, two strain gages were installed 180 degrees apart on the free length of the anchor rod between the concrete foundation and the underside of the base plate.

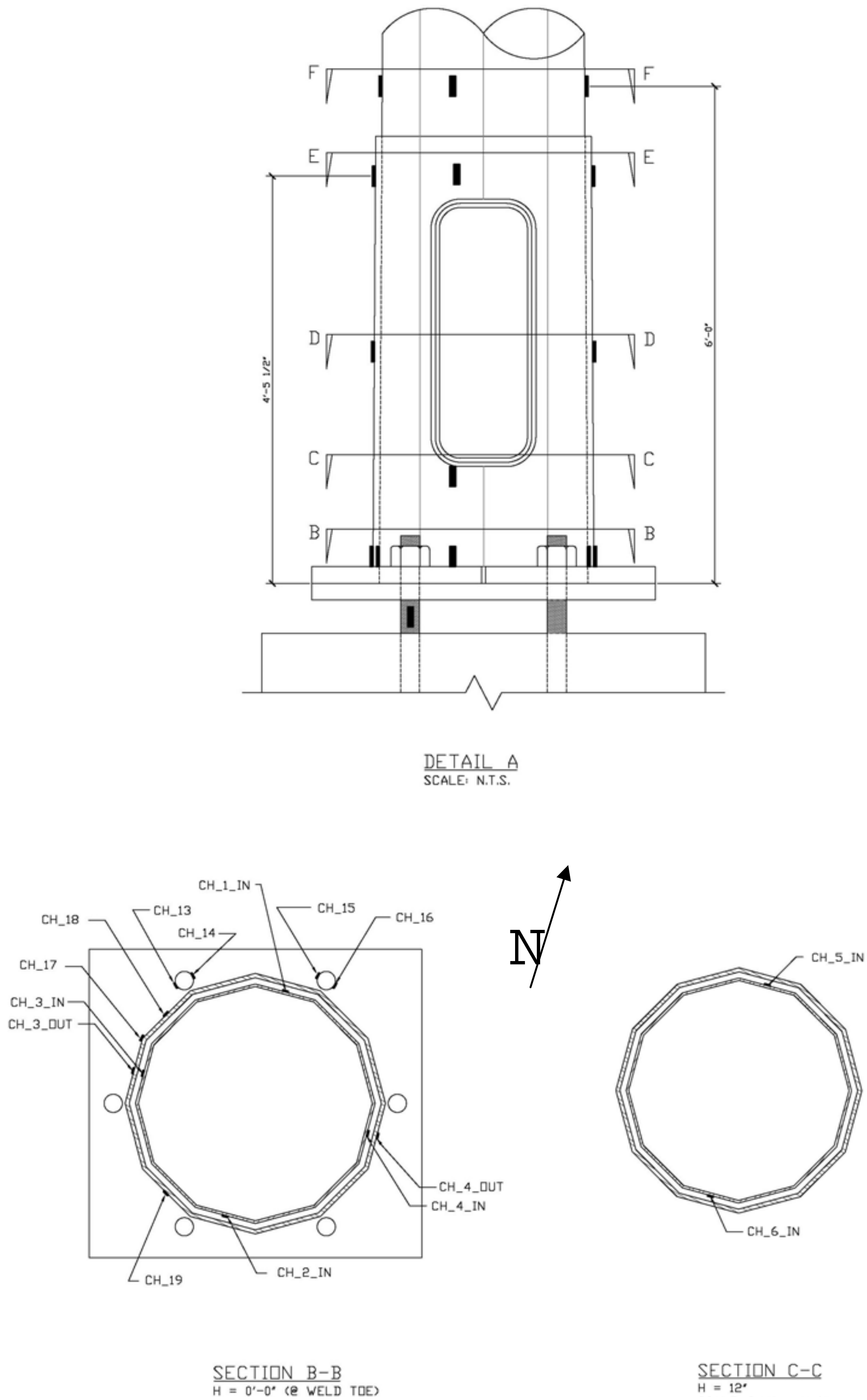

Figure 1. Section drawings showing strain gage locations

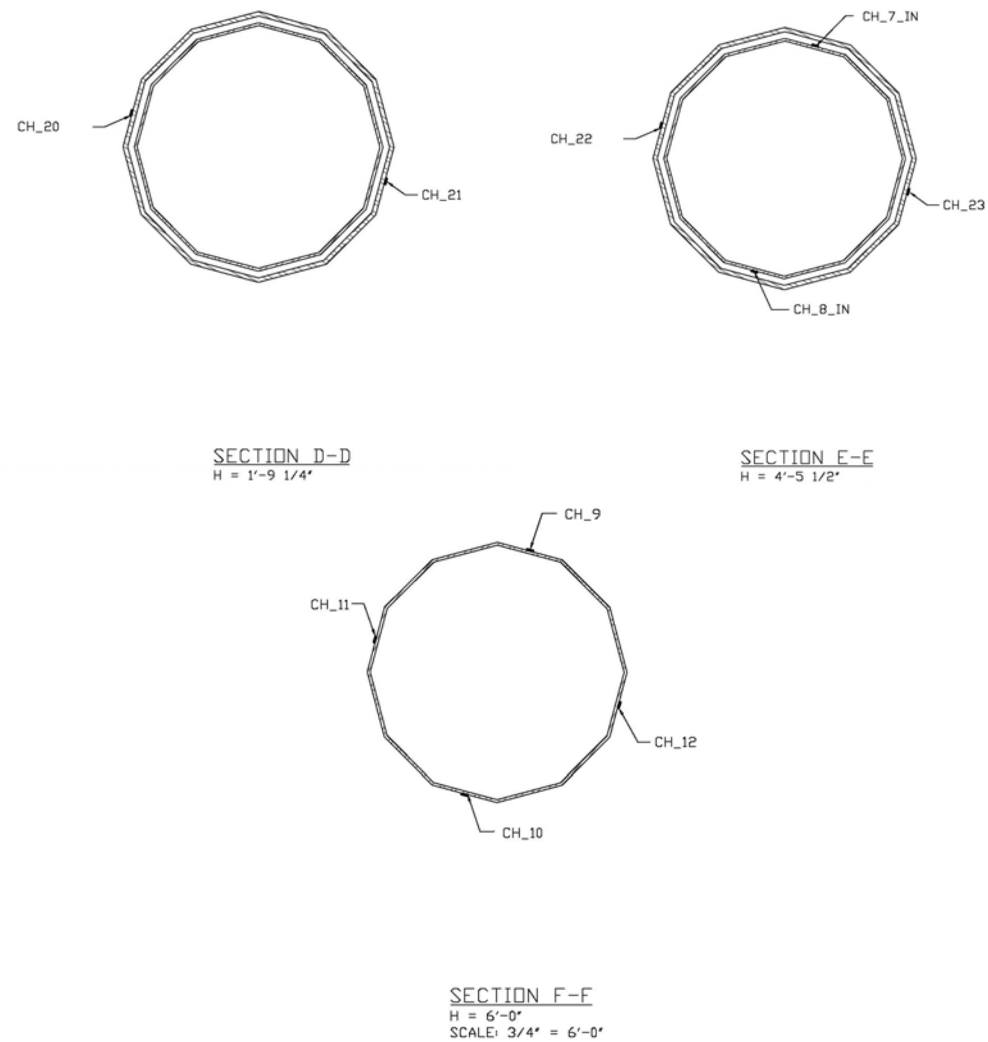

Figure 1. Section drawings showing strain gage locations (continued)

RESULTS OF STATIC TESTING

Static tests were performed on the high-mast tower, both with and without the reinforcing jacket in place. In both cases, the load was applied in both the north and west directions. Tests were repeated multiple times. The load was applied using a nylon sling wrapped around the tower at a height of approximately 35 ft. The load was applied using a come-along connected to the sling with a wire rope. The other end of the wire rope was connected to the back of a heavy truck at ground level. This is shown schematically in Figure 2. As a result of the inclination of the cable, a lateral force (causing bending) and a downward force (causing compression) are applied to the tower.

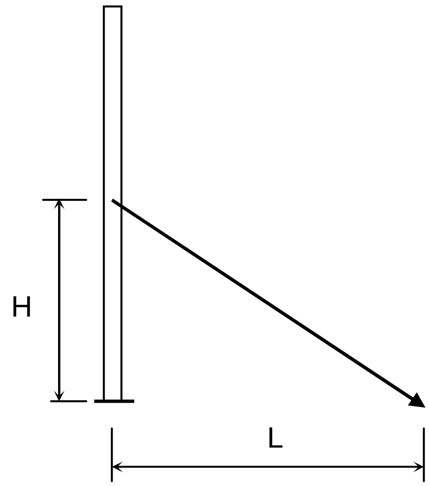

Figure 2. Schematic drawing of static pull tests with key dimensions

Presented in Table 1 are the peak stresses measured in all strain gages on the as-built tower (no reinforcing jacket) for pull tests in the north-south directions, respectively. Also shown in the table are the calculated moment and stress at a section 6 ft. above the baseplate. This is compared to the measured bending stress, calculated by taking the half the difference between the peak stresses measured at the two gages on opposite sides of the tower in line with the load. The measurements from strain gages CH_9, 10, 11, and 12 are used for this calculation. For north-south loading, the bending stress is equal to (CH_10 – CH_9)/2. For east-west loading, the bending stress is equal to (CH_12 – CH_11)/2. It can be seen that there is good agreement between the calculated and measured bending stress. Similar agreement was observed in the east-west pull tests.

Table 2 contains the peak measured stresses in all strain gages on the retrofitted tower (with reinforcing jacket) for the pull tests in the north-south direction. Again, it can be seen that there is good agreement between the calculated and measured bending stress.

In the jacket-reinforced tower, it can be seen that, at the base of the tower section (CH_1_IN, CH_2_IN, CH_3_IN, and CH_4_IN), significant stress levels remain in the tower, though they are markedly reduced from the as-built case.

Table 1. As-built tower, stresses measured during static north-south pull tests

Data Channel	Location	NS Pull				Unit
		Pull_1	Pull_2	Pull_3	Pull_4	
CH_1_IN	N face tower at base	-2.60	-3.39	-6.40	-6.07	ksi
CH_2_IN	S face tower at base	2.23	3.28	6.53	6.51	ksi
CH_3_IN	W face tower at base	-0.19	-0.17	0.15	0.05	ksi
CH_4_IN	E face tower at base	-0.09	0.12	-0.12	0.29	ksi
CH_5_IN	N face tower 12" above base	-1.41	-1.74	-3.25	-2.97	ksi
CH_6_IN	S face tower 12" above base	1.24	1.95	3.86	3.89	ksi
CH_7_IN	N face tower 53.5" above base	-1.43	-1.77	-3.35	-3.02	ksi
CH_8_IN	S face tower 53.5" above base	1.16	1.84	3.62	3.75	ksi
CH_13	NW anchor rod	-0.81	-0.98	-1.60	-1.51	ksi
CH_14	NW anchor rod	-1.72	-2.22	-4.01	-3.87	ksi
CH_15	N anchor rod	-1.21	-1.46	-2.83	-2.52	ksi
CH_16	N anchor rod	-2.14	-2.79	-5.39	-5.07	ksi
CH_9	N face tower 6' above base	-1.30	-1.59	-3.09	-2.72	ksi
CH_10	S face tower 6' above base	1.02	1.57	3.13	3.17	ksi
CH_11	W face tower 6' above base	-0.11	-0.07	0.18	0.21	ksi
CH_12	E face tower 6' above base	-0.14	0.09	-0.04	0.27	ksi
	Load	0.78	1.06	1.83	2.02	kips
	Moment @ 6' above baseplate	19.82	26.82	46.44	51.40	k-ft
	Calculated Stress	1.18	1.59	2.76	3.05	ksi
	Measured Bending Stress	1.16	1.58	3.11	2.94	ksi
	Meas./Calc'd Stress	0.99	0.99	1.13	0.96	ksi

H= 35', L = 63.6'

Table 2. Retrofitted tower, stresses measured during static north-south pull tests

Data Channel	Location	NS Pull Pluck_A	NS Pull Pluck_B	NS Pull Pluck_C	Units
CH_1_IN	N face tower at base	-2.48	-0.99	-1.70	ksi
CH_2_IN	S face tower at base	2.93	4.34	3.05	ksi
CH_3_IN	W face tower at base	0.61	2.01	1.01	ksi
CH_4_IN	E face tower at base	0.49	1.97	1.02	ksi
CH_5_IN	N face tower 12” above base	-0.66	0.76	-0.08	ksi
CH_6_IN	S face tower 12” above base	1.62	3.11	1.99	ksi
CH_13	NW anchor rod	-2.59	-1.01	-1.86	ksi
CH_14	NW anchor rod	-2.56	-1.54	-2.23	ksi
CH_15	N anchor rod	-1.10	0.36	-0.50	ksi
CH_16	N anchor rod	-5.44	-4.39	-4.67	ksi
CH_9	N face tower 6’ above base	-2.56	-1.06	-1.93	ksi
CH_10	S face tower 6’ above base	3.05	4.92	3.47	ksi
CH_11	W face tower 6’ above base	0.53	2.09	1.00	ksi
CH_12	E face tower 6’ above base	0.37	2.01	0.97	ksi
CH_3_OUT	W face jacket at base	0.41	1.64	0.70	ksi
CH_4_OUT	E face jacket at base	0.38	1.77	0.90	ksi
CH_17	WNW face jacket at base on bend	-1.26	0.08	-0.87	ksi
CH_18	WNW face jacket at base	-1.22	-0.29	-1.03	ksi
CH_19	SSW face jacket at base	3.11	4.74	3.36	ksi
CH_20	W face jacket 21.25” above base	0.30	1.78	0.69	ksi
CH_21	E face jacket 21.25” above base	0.18	1.46	0.55	ksi
CH_22	W face jacket 53.5” above base	0.29	1.36	0.59	ksi
CH_23	E face jacket 53.5” above base	0.36	2.02	0.70	ksi
	Load	1.95	1.98	1.89	kips
	Moment @ 6' above baseplate	49.56	50.22	47.99	k-ft
	Calculated Stress	2.94	2.98	2.85	ksi
	Measured Bending Stress	2.81	2.99	2.70	ksi
	Meas./Calc'd Stress	0.95	1.00	0.95	ksi

H= 35’, L = 63.6’

PRELIMINARY RESULTS OF LONG-TERM MONITORING

Stress-range histograms were recorded every ten minutes using the rainflow cycle counting algorithm (Miner 1945). These histograms were generated for nine selected strain gages. A stress-range histogram is basically a tally of stress cycles of predetermined ranges. Every ten minutes, the data acquisition system updates the tally. A fatigue evaluation of the towers was performed using the stress-range histograms, which were truncated at a level equal to approximately 1/4 of the constant amplitude fatigue limit (CAFL) of the detail in question per AASHTO. That is, all cycles with stress ranges less than the truncation level were removed from the histogram prior to calculation of the effective stress.

In addition to the stress-range histograms, stress time history data were recorded when predefined trigger events occurred. These trigger events occurred when wind speed and stress events at selected locations exceeded various levels. When a trigger event was detected, data were recorded from all sensors for a predefined length of time. The stress time history data were used to assess the validity of large stress-range cycles recorded in the stress-range histograms and to understand the wind phenomena that caused them.

Finally, average wind data were recorded continuously, on five-minute intervals. During each interval, the data logger records the average and maximum wind speed and the average wind speed.

A total of nine strain gages are currently being monitored, as identified in Table 3 and as shown on the instrumentation plan Figure 1. A fatigue life estimate was performed for each of the gages listed in Table 3, using the stress-range histograms developed for the period between November 8, 2006, through February 13, 2007.

Table 3. Summary of strain gages for which stress-range histograms were developed

Strain gage	Location
CH_1	N side on tower; at base weld centered on face
CH_3IN	W side on tower; at base weld centered on face
CH_3OUT	W side on jacket; at base weld centered on face
CH_9	N side on tower; 6’ above baseplate centered on face
CH_11	W side on tower; 6’ above baseplate centered on face
CH_14	NW anchor rod
CH_16	N anchor rod
CH_17	W side on jacket; at base weld on bend
CH_23	E side on jacket, 4’-5 1/2” above baseplate

The strain gages on the tower beneath the reinforcing jacket adjacent to the full-penetration groove weld (strain gages CH_1 and CH_3IN) are considered Category E, due to the fact that the backing bar is not welded to the base plate by a full-penetration weld. (It is noted that all fatigue categories cited herein are per current AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Supports). The strain gages on the reinforcing jacket adjacent to the base weld (CH_3OUT and CH_17) are considered Category E as a full-penetration groove weld. The strain gages on the anchor rods (CH_14 and CH_16) are considered Category D based on the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Supports, Table 11-2. Strain gage CH_23 on the reinforcing jacket some distance above the baseplate is considered Category B as a bolted connection. Strain gages above the jacket on the tower (CH_9 and CH_11) are also considered Category B as a bolted connection.

Table 4. Summary of fatigue life calculations (November 8, 2006, through February 13, 2007)

Strain gage	Assumed category (CAFL)	S_{Rmax} (ksi)	Cycles > CAFL #	Cycles > CAFL %	S_{Reff} (ksi)	Cycles/ day	Remaining life (years)
CH_1	E' (2.6)	8.5	10,849	0.21%	1.0	54,288	17
CH_3IN	E' (2.6)	9.0	9,738	0.15%	1.0	65,225	15
CH_3OUT	E (4.5)	11.0	574	0.02%	1.6	29,732	18
CH_9	B (16)	13.0	0	0.00%	6.0	27	infinite
CH_11	B (16)	12.5	0	0.00%	5.8	25	infinite
CH_14	D (7)	21.6*	621	0.04%	2.2	16,465	30
CH_16	D (7)	16.0	2,796	0.11%	2.4	25,651	11
CH_17	E (4.5)	25.8*	58,238	0.83%	2.2	72,665	4
CH_23	B (16)	8.0	0	0.00%	5.0	1	infinite

*Maximum stress cycle determined from review of time-history data

Review of the stress-range histograms revealed that very high-stress cycles were recorded in a number of the strain gages, in particular gages CH_14 and CH_16 on the anchor rods and CH_17 at the bend line on the west face of the reinforcing jacket. The effect of these higher stress cycles are reflected in the relatively low fatigue life estimates presented in Table 4. However, it should be noted that the strain gages at the base were installed adjacent to the weld to at the baseplate weld. As a result, the strain gages in their current position are located in an area of high stress gradient. Since the AASHTO S-N curve is calibrated to be used with nominal stresses, the life estimates presented in Table 4 will likely underestimate the actual fatigue life. (It is noted that the fatigue categories used for this evaluation will be updated based on the results of the Iowa Department of Transportation –funded laboratory testing of retrofitted high-mast towers, currently underway at Purdue University.)

The highest stress cycles recorded during the monitoring period were the result of a wind storm that occurred on November 10, 2006, at approximately 7:10 a.m. The average wind speed at the time was only approximately 20 mph, with gusts up to 33 mph. The winds were from the north (note, the data in Figure 3 is an uncorrected wind direction; 58 degrees must be added to each measurement to correct to the compass directions). The stress cycles resulting from this wind event caused a second peak in the stress-range histograms at many locations where gages were installed.

Though the stress magnitudes and corresponding fatigue life estimates may be alarming, the frequency of occurrence of this wind event must be evaluated through continued monitoring. It must be noted that the fatigue life estimates are based on the assumption that the number and magnitude of stress cycles measured during the three-month period presented are the same for all three-month periods for the remainder of the tower life. This is most likely not the case, as this event may have been a rare occurrence. This will be evaluated using the remaining long-term data.

At this time, the authors believe that the tower was excited by an initial gust (see Figure 3) that caused the tower to begin vibrating in its first mode. The first mode frequency for this pole is approximately 0.3 Hz (or a period of vibration of 3.3 seconds). This means that the time it takes for the pole to start at a positive peak stress, vibrate to peak negative stress, and return to peak positive stress is 3.3 seconds.

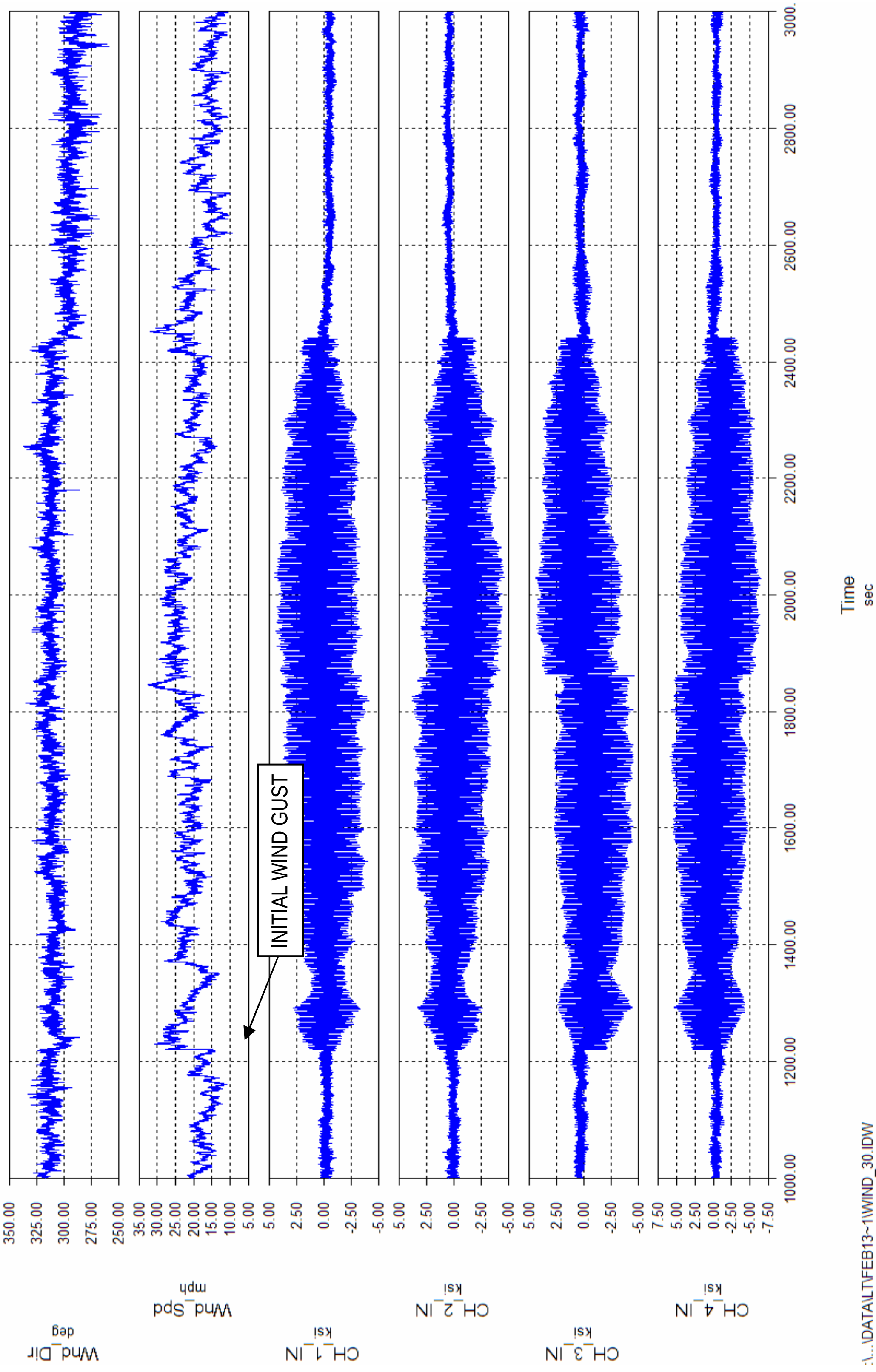

Figure 3. High-stress event, November 10, 2006, 7:10 a.m.

This observed vibration is most likely not vortex shedding, since the critical velocity for vortex shedding in the first mode is significantly less than the observed 20 mph present during the event. The wind gusts appear to have been in phase with the first-mode natural frequency of the tower, causing the magnitude of vibration to increase. This will be studied further as part of this project to fully assess the nature of this phenomena and its effect on the fatigue performance of the retrofitted and unretrofitted high-mast towers.

Figure 4 contains a close-up stress time history for the four strain gages on the tower above the reinforcing jacket (6 ft. above the base plate). It can be seen that the time it takes the stress to vary from peak positive to peak negative and back to peak positive stress is approximately three seconds. Furthermore, strain gages CH_9 and CH_11 are in phase. Strain gages CH_10 and CH_12 are also in phase with each other, but collectively out of phase by 180 degrees with CH_9 and CH_11. Note that the magnitudes are roughly equal. This indicates that the neutral axis of bending lies on a 45 degree line running between CH_9 and CH_11 on one side and CH_10 and CH_12 on the other (i.e., running northeast-southwest). A similar plot for the four gages on the tower at the base weld (beneath the reinforcing jacket) is shown in Figure 5. It can be seen that the same direction of bending is causing the measured stresses.

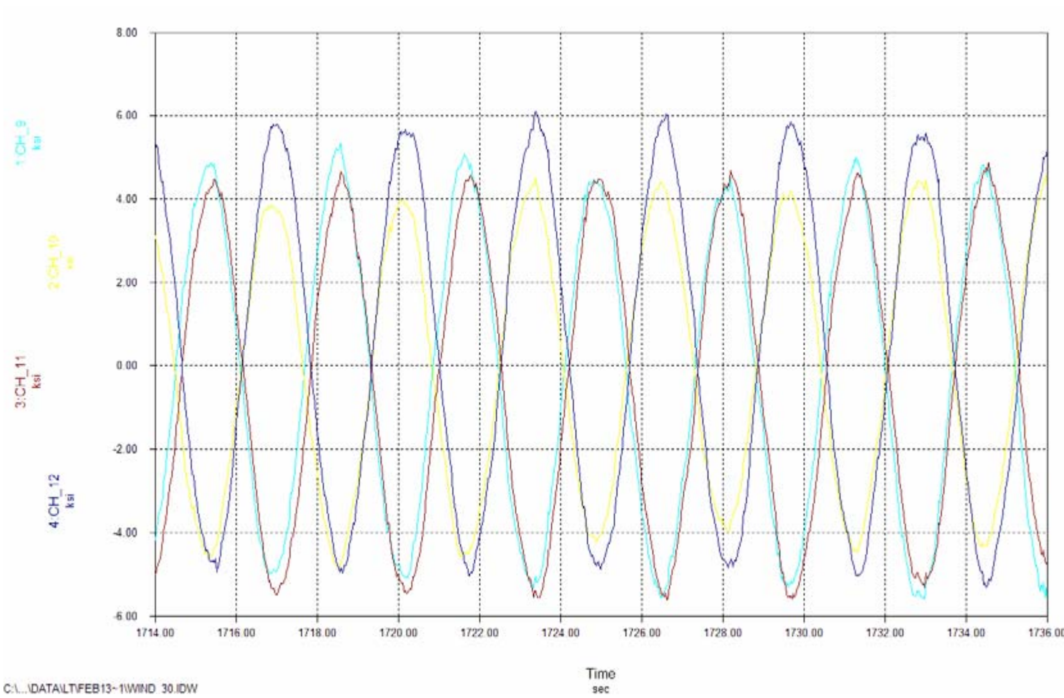

Figure 4. High-stress event, November 10, 2006, 7:10 a.m.; strain gages six ft. above the baseplate

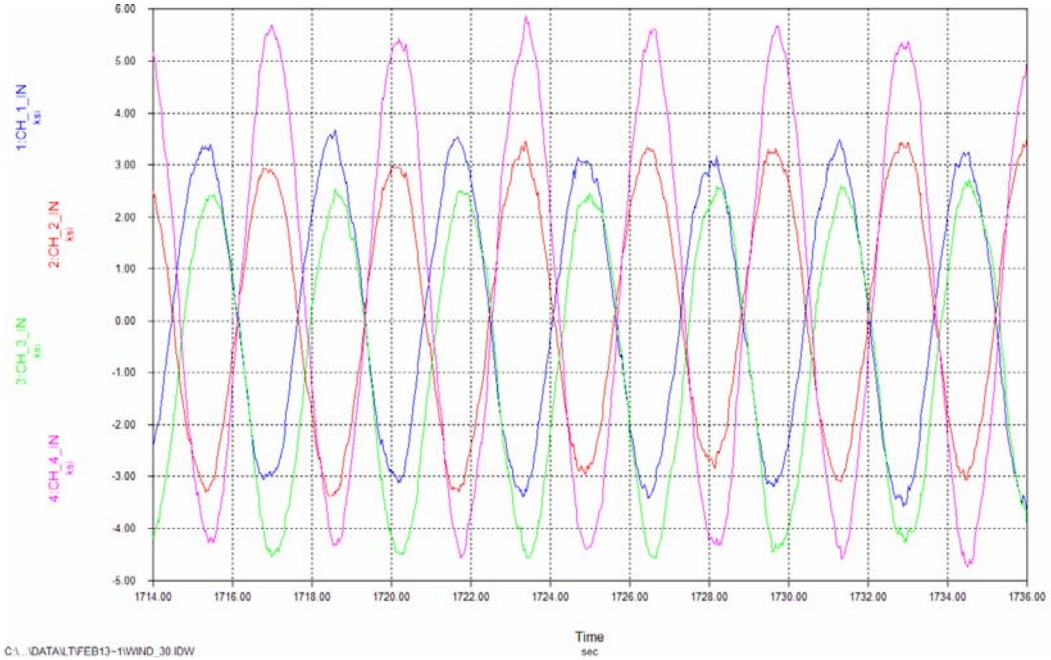

Figure 5. High-stress event, November 10, 2006, 7:10 a.m.; strain gages on the tower at the base weld

FIELD INSPECTION OF HIGH-MAST LIGHTING TOWERS

Based on the results of this study and the observed causes of other failures of high-mast lighting towers, it is clear that these structures should be regularly inspected. There is no doubt that these are fracture-critical structures and that collapse can be extremely hazardous. Below are some recommendations related to the inspection of these towers:

- The bottom sections of high-mast towers should be inspected visually at the base weld and handhole on a regular interval. The interval between inspections should be related to the susceptibility of the structure to fatigue cracking. This susceptibility can be determined by evaluating the pole using current specifications. Recent inspection experience by several owners indicates that cracks in weathering steel towers can be very difficult to detect visually. The use of magnifying glasses, grinders, and magnetic particle/dye penetrant testing will likely be required to ensure a thorough inspection.
- The risk of anchor rod fatigue is also critical. If it is established that there is a potential for anchor rod cracking, ultrasonic testing of the anchor rods should be performed during every inspection. In addition, special attention should be given to ensuring that both the upper and lower anchor nuts are tight. It would be prudent to check existing structures using a torque wrench or other method.
- Weathering steel poles are particularly sensitive to the collection of debris inside and outside of the tube. Inspections should also ensure that leaves, soil, or other debris are not building up on the outside of the pole and that there is ample clear space around the base. Inspections of the inside of the base section must also be conducted to ensure corrosion is not taking place on the inside of the tube wall. Furthermore, inspections must also remove any debris buildup from the inside of the pole. Removal of grout between the base plate and the footing is beneficial in helping keep the area dry and free of debris. Special attention must be given to section loss or any poles located adjacent to the highway that may be susceptible to salt splash.

CONCLUSIONS

The following preliminary conclusions are made:

1. Static pull tests were performed on the high-mast tower in both the as-built and jacket-reinforced condition. The stresses in the tower were reduced by the presence of the jacket, but not eliminated, and in fact they are still significant.
2. Through the long-term monitoring, which continues, a large number of high-amplitude stress cycles were measured in the tower and reinforcing jacket. Initial fatigue life estimates are believed to be conservative due to the reasons cited in this paper. Revised detail categorization will be based on the results of the ongoing laboratory fatigue investigation of high-mast towers at Purdue University.
3. During the monitoring, an unusual and likely infrequent wind event that occurred in November 2006 induced most of these high-amplitude stress cycles. This type of behavior was not observed during the long-term monitoring of the same tower conducted in 2004–2005. It is surprising that the peak wind speed recorded during this event was only 33 mph. However, the frequency of the in-line wind gusting appears to have matched the natural frequency of the tower, thus magnifying the vibration amplitudes. The frequency of occurrence of this type of wind event must be evaluated. (Currently, the presence of these cycles in the spectra will skew the life estimates if the event rarely occurs.)

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