Investigation of the September 24, 2002, Collapse of the 1965-foot High KDUH-TV Antenna Tower in Hemingford, Nebraska

U.S. Department of Labor Occupational Safety and Health Administration Directorate of Construction

March 2003



REPORT

Investigation of the September 24, 2002, Collapse of the 1965-foot High KDUH-TV Antenna Tower in Hemingford, NE

Report prepared by Mohammad Ayub, PE Dinesh Shah, PE

REPORT

Background:

The Directorate of Construction, National OSHA Office, was requested to provide assistance in the investigation and causal determination of the September 24, 2002, collapse of the 1965-foot high KDUH-TV Tower in Hemingford, NE. The incident occurred when workers were replacing the diagonals of the sections of the tower. Two structural engineers from the Office of Engineering, Directorate of Construction, accompanied by personnel from the Omaha OSHA Area Office, visited the incident site on September 30, 2002. One of the structural engineers also visited the storage yard at Scottsbluff, NE, at a later date where the selected retrieved sections of the collapsed tower were stored.

Incident:

The incident occurred around 11:45 a.m. as a work crew was replacing the existing diagonals with diagonals of larger diameter in the panels of section 10 and the bottom panel of section 47. The entire project consisted of upgrading the existing KDUH-TV antenna tower to support the load of a new high-definition TV antenna and to replace aviation lights. The tower is located in a remote field, 22 miles northwest of Alliance, NE. There were two fatalities and three injuries when the tower collapsed. The collapse crushed a nearby car and pick up truck. A small grass fire, ignited by electrical wires severed in the collapse, was quickly extinguished after rescue crews arrived.

Description of the Project:

The 1965-foot high KDUH-TV antenna tower was under contract to replace certain tower diagonals and struts to support the high-definition TV antenna and to replace aviation lights as mandated by the Federal Aviation Administration (FAA). The owner of the tower was Duhamel Broadcasting Enterprises of Rapid City, SD, which used the tower to broadcast signals from KDIJH-TV in Scottsbluff, NE. Structural Systems Technology, Inc. (SST) of McLean, VA, was the structural engineer of record for the tower modification. The subcontract for roofing work on the building at the tower's base was awarded to Weathercraft Roofing Co. of Scottsbluft NE. This work was nearly completed prior to tower collapse. Mid Central Tower of Illinois (Mid Central) was under contract to replace the diagonal and strut members of the tower.

The tower, triangular in plan, was composed of legs, guys, struts, diagonals, and redundant members. The tower consisted of 63 sections, with each 30-foot high section composed of three 10-foot high panels. The sections were numbered in ascending order from the top to the base. The top section was numbered as 1; the bottom section was numbered 63. The panels of each section were numbered in ascending order from the bottom to the top. The bottom, middle, and top panel of the sections were numbered as 1, 2 and 3, respectively. The tower was guyed at the three corners of the triangle at eight elevations of 218'-6'', 428'-6'', 648'-6'', 878'-6'', 1118'-6'', 1358'-6'', 1608'-6'', and 1875'-6'' (figure 1). The modification of the tower (figures 2, & 3))

consisted of:

- Adding redundant members (double angles 2 x 2 x ¹/₄) to section 1, panels 2 & 3
- Replacing existing 1" diameter diagonals with new 1¹/₈" diameter diagonals in section 10, panels 1 & 2
- Replacing existing $\frac{3}{4}$ " diameter diagonals with new $\frac{7}{8}$ " diameter diagonals in section 47, panel 1

Collapse Description:

The antenna tower (Figure 1) was originally erected in 1968. To support the new High Definition TV antenna, the structural engineer of record determined that certain structural members be replaced. The tower had been previously modified in 2001 by replacing guy wires with larger guy wires. The new modifications required replacement of the structural members in sections 1, 10, & 47. The subcontractor, Mid Central, planned to begin work simultaneously on lower section 47, and upper section 10; and later on top section 1. On September 24, 2002, two employees of Mid Central were working at the site, one on section 10 and another at section 47. An employee of the Weathercraft Roofing Co. was working at the base of the tower. The tower collapsed around 11:45 a.m., when the workers were replacing structural members at section 10 and 47, killing the two workers of Mid Central. The third Weathercraft Roofing Co. employee on the ground suffered minor injuries.

The tower fell apart in three pieces. The two base sections remained standing vertically. The next 14 sections (section 62 through 48) fell in one piece. The remaining sections fell in a shape (figure 4). Section 47 was the most distressed and was twisted and crumpled into the ground. Other sections were also twisted, tangled, and deformed to varying degrees. During the clean up operation, section 47 had to be cut into pieces and could not be salvaged for later examination. Wind data from National Weather Report indicated the wind speed at the time of collapse was approximately 8 miles per hour.

Selected sections of the collapsed tower were removed and stored in the warehouse of R & C Welding of Gering, NE, for further technical evaluation. OSHA representatives examined sections 10 and 48 in the warehouse. Our conclusions are identified in figures 5 and 6.

Field observations indicated that the bolts connecting the, diagonals and redundant members, hereafter called lug bolts, were missing in a number of panels of sections 10, 47 and 48. Visual examination indicated that the lug bolts were missing prior to the collapse because the lug boltholes did not exhibit any deformation and/or elongation normally expected if they had failed during collapse. The lug boltholes appeared "clean".

In panel 1 of section 10, all six diagonals were replaced with new diagonals and bolted to the gusset plates at each end by high strength bolts, as required by the construction documents. However, the lug bolts were not placed in any of the three faces of panel 1. In panel 2 of section 10, four out of six diagonals were replaced with new diagonals and bolted at each end. Two old

diagonals remained. A few of the bolts connecting the old diagonals sheared off during the collapse (see Figure 6). Again, the lug bolts were missing in all three faces of panel 2, including the face where the existing diagonals had not yet been replaced.

Of all the sections of the failed tower, Section 47 was the most distorted and wrinkled, and partially buried in the ground. However, panel 1 of section 47 was examined though with difficulty due to distortions of its members. Examination at the scene revealed that the lug bolts of panel 1 were missing on all three faces as the holes were devoid of any deformations or elongations. Also, observation of section 48 in the warehouse indicated that the lug bolts were missing on two faces of panel 3 of section 48. Lug boltholes did not exhibit any sign of distortion or deformation that led to our conclusion that the bolts were missing prior to the collapse. It was also discovered that where panel 3 of section 48 joined panel I of section 47 at least one diagonal and one strut member were unbolted at the gusset plate connection of the legs of section 48. Figure 5 indicates the missing bolts and missing members. As the removal of the bolts of the diagonal and the strut was critical to the stability of the tower and this investigation, the OSHA Salt Lake Technical Center (SLTC) visually examined the holes to confirm whether the bolted connections of the diagonal and strut were removed prior to the collapse. A metallurgist from SLTC visited the warehouse and examined the holes in question of section 48 confirming that the connections of a diagonal and a strut member at section 48 were removed prior to the collapse (see appendix A for SLTC report).

See figures 7 to 27 for the collapsed tower.

Analysis:

Professor Hugh Bradburn of the Department of Civil Engineering, University of South Carolina, Columbia, SC, was contracted to perform the following structural analyses of the antenna tower:

- 1. To determine the adequacy of the structural design of the KDUH-TV tower with new diagonals and new guy wires, as specified by the structural engineer of record, to support the new loads in accordance with the applicable TIA/EIA standards for steel antenna towers.
- 2. To perform structural analyses of the KDUH-TV tower with the conditions that existed immediately prior to the collapse of the tower, as best determined by OSHA, i.e. with bolts and members in certain tower sections removed.

A three dimensional finite element program based on a stiffness matrix was used to model and analyze the structure. The program recognizes large displacements and large rotations of the members whose shears and moments are computed in deformed positions. The guys are modeled by three dimensional, geometrically nonlinear finite element cable stiffness elements. The tower is divided into elements corresponding roughly to a tower segment. Truss freedoms of each element are reduced and transformed to equivalent beam type elements, then assembled into a tower stiffness matrix. The program reduces compression in slender diagonals instead of eliminating them completely which produces a much more realistic distribution of forces in the

structure. This program was written, developed and run by Dr. Hugh Bradbum. Many nationally recognized engineering firms involved in the analysis and design of cable supported tower structures have verified the validity of these computer programs.

Five analyses were performed, as follows:

- To determine the gross results with existing members with no wind;
- To determine member forces in the tower with existing members with no wind
- To determine gross result with the proposed new members of the tower with a wind pressure of 65 psf;
- To compare maximum member force with allowable force for all tower members (based on AISC/ASD) with wind pressure of 65 psf;
- To determine member forces in the tower with proposed new members with wind pressure of 65 psf.

The applicable design standard for the tower was considered to be TIA/EIA RS-222-C. The objective of the analysis was to determine causal factors that may have contributed to the collapse of the antenna tower and to determine whether the HDTV antenna loads would have overstressed the tower section members when completed.

The properties of the tower members of interest were taken from original contract documents prepared by SST of McLean, VA. It was assumed that the drawing reflected as-built conditions so no verification of the member sizes and properties was done except in isolated locations.

The wind speed was taken as 65 mph and the resulting wind pressure was uniformly applied for the entire height, as recommended by the EIA standard. Wind was applied in three directions, direction 1 is normal to a tower face; direction 2 is 180 degrees from direction 1; and direction 3 is parallel to a tower face.

The following documents provided the basis for the structural analyses:

- a) Contract drawings dated 1977, 1993, and 2002.
- b) A report "Rigorous Computer Structural Analysis and Evaluation of the KDUH-TV 1965-foot Guyed Tower Hemingford, Nebraska" prepared by SST, structural engineer of record.
- c) Miscellaneous photographs and newspaper articles concerning the collapse.
- d) Weather data for Alliance, NE, for September 23.24, 2002.
- e) Miscellaneous information.

Appendix B contains the report on the structural analysis

Prior to the tower collapse, the workers were replacing existing diagonals with new diagonals of larger diameter in sections 10 and 47. National Weather Report data indicates that the wind speed at the time of collapse was approximately 8 miles per hour, considered inconsequential.

Towers are designed to carry gravity, wind, and ice loads. All members, legs, diagonals, struts and redundant members provide structural stability to the tower with little redundancy. Absence or removal of a member can trigger catastrophic failure unless a substitute is provided before removal. Legs carry axial loads and derive their strength from shorter unbraced lengths by the presence of redundant members and their connection to the diagonals with lug bolts. If the lug bolts are removed without providing any substitute to the frame, the leg is subjected to a higher unbraced length that immediately reduces its load carrying capacity. If a diagonal is also removed in addition to the lug bolts, the unbraced length of the leg is greatly increased, jeopardizing its load carrying capacity. Failure of one leg can result in a tower collapse because there is no redundancy. Redundant members and diagonals are critical to the stability of the tower.

The Structural Engineer of Record (SER) had stated that he had provided instructions (Appendix C) to the contractor to use Come-A-Longs before removing diagonals. It is believed that the Come-A-Longs were not used. The diagonals cannot be removed without removing the center lug bolts. There is a presumption in the instructions issued by the SER that removing the lug bolt will not substantially reduce the load carrying capacity of the leg. Structural analysis indicated that for the KDUH-TV tower, such was the case.

Section 10:

Structural analyses indicated that the actual load carried by each leg at section 10 was approximately 90 kips at the time of the incident. As mentioned above, wind was not considered a factor. The capacity of the legs was determined to be far greater than 90 kips. If the lug bolts of section 10 were removed on all three faces without disconnecting the diagonals at their ends, the capacity of the legs would be reduced but would be more than adequate to support the loads. However, if one diagonal was removed in addition to the lug bolts, the capacity is significantly reduced to between 137 and 150 kips. This reduced capacity is still higher than the actual loads carried by the legs. It was therefore concluded that the lack of use of Come-A-Longs in replacing the diagonals was not critical to Section 10.

Section 47:

The actual loads carried by the legs at section 47 was determined to be 260 kips without wind. The capacity of each leg was determined to be 970 kips which provided adequate factor of safety. However, if the lug bolts were removed, the capacity of the legs would be reduced to 690 kips, based upon inelastic buckling, but still well above the actual load. If a diagonal on one face is removed in addition to the lug bolts, the capacity will be significantly reduced to between 260 and 285 kips, equal or close to the actual load the leg must support. With such conditions, the collapse of the tower could be imminent. If a horizontal strut member was removed in addition of a diagonal, as was the case in this collapse, the failure would be inevitable.

An analysis of the proposed upgrade was performed using Telecommunications Industry Association (TIA) and Electronic Industry Association (EIA) Standard RS-222-C for a uniform wind pressure of 65 pounds per square foot with no ice. The results of this analysis indicated that the proposed upgrade met all requirements of EIA Standard RS-222-C

Conclusions:

Based upon the above findings, it is concluded that:

- Construction activities at tower section 47 immediately before the incident caused the tower collapse. Simultaneous construction activities at section 10 did not contribute to the collapse.
- A worker removed the bolts connecting the diagonals and redundant members of panel 1, Section 47, and a diagonal of one face of panel 1, Section 47, resulting in a severe reduction of load carrying capacity of the tower legs, which eventually led to the tower collapse. The worker also removed a horizontal strut of panel 1, Section 47 that contributed to the failure.
- Removing the bolts at the intersection of the diagonal and the redundant members of panel 1, Section 47, would not have been critical to the structural integrity of the tower. However, the tower legs in Section 47 lost their capacity to support the loads when the diagonal was also removed.
- 4. Though similar construction procedures were used at Section 10, the capacity of the legs of Section 10 did not diminish to the extent that they were unable to support the loads.
- 5. Come-A-Longs or other devices were not used prior to removing the diagonal. The contractor did not follow the generally accepted industry practice of installing a temporary bracing member before removing a diagonal. The structural engineer of record stated that he provided instructions to the contractor to use a Come-A-Long. It is, however, believed that the structural engineer did not adequately warn the contractor of the critical nature of the diagonals to the stability of the entire tower.
- 6. Wind was not a causal factor in the tower collapse.
- 7. The proposed design by the structural engineer of record was determined to be adequate to support new loads.





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DIAGONAL SCHEDULE

TOWER SECTION	PANEL	DIAGONALS `	BOLTS							
10	1	"D1" (1 1/8"ø)	(2) 7/8"ø x 2 1/4" EA. END							
10	2	"D2" (1 1/8"ø)	(2) 7/8"ø x 2 1/4" EA. END							
47	1	"D3" (7/8"ø)	(2) 5/8"ø x 1 3/4" EA. END							

FIGURE 3

GENERAL NOTES

1.1.

- 1. THE SUCCESSFUL ERECTION CONTRACTOR SHALL PROVIDE PROOF OF EXPERIENCE AND CAPABILITY IN UNDERTAKING A PROJECT OF THIS NATURE.
- 2. THE ERECTION CONTRACTOR SHALL HAVE BOTH THE ASSEMBLY AND FABRICATION DRAWINGS TOGETHER BEFORE STARTING WORK.
- 3. ALL BOLTED FIELD CONNECTIONS SHALL BE MADE WITH A.S.T.M. A325 GALVANIZED HIGH STRENGTH STEEL BOLTS, HARDENED WASHERS AND ANCO LOCKNUTS.
- 4. ANY BOLT HOLE MISMATCHING SHALL BE CORRECTED BY REAMING, THEN A COAT OF ZINC-RICH PAINT (ZRC, GALVANOX OR EQUAL) SHALL BE APPLIED.
- 5. THESE DRAWINGS INDICATE THE MAJOR OPERATIONS TO BE PERFORMED, BUT DO NOT SHOW EVERY FIELD CONDITION THAT MAY BE ENCOUNTERED. THEREFORE, PRIOR TO BEGINNING WORK, THE CONTRACTOR SHOULD SURVEY THE JOB THOROUGHLY TO MINIMIZE FUTURE FIELD PROBLEMS.
- 6. UPON COMPLETION OF ALL WORK, THE SITE SHALL BE CLEANED OF ALL DEBRIS AND ANY SURPLUS MATERIALS, NOT REMOVED FROM THE SITE, SHALL BE NEATLY STORED IN AN AREA DESIGNATED BY THE INSPECTOR.







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FIGURE 7 (SECTION 10)



FIGURE 8



FIGURE 9 (SECTION 47)







FIGURE 11



FIGURE 12



REPORT OF ANALYSIS OF BOLT HOLES IN KDUH TELEVISION TOWER

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February 18, 2003

REPORT OF ANALYSIS OF BOLT HOLES IN KDUH TELEVISION TOWER

On 24 September 2002, the KDUII Television tower near Scottsbluff, NE failed and fell while being retrofitted. The failure resulted in two fatalities and several other casualties. Pertinent sections of the tower were retained and stored in Gering, NE. I examined those remaining structures to describe the condition of a number of bolt holes.

The structure was a 1965 feet tall, three sided guyed communications tower, with each side approximately 10 feet wide. The tower was constructed of sections, each 30 feet tall. These sections were numbered sequentially from the top. Each three-sided section consisted of three 10-foot tall panels per face. Each panel consisted of two diagonal cross member trusses, a horizontal redundant member across the face at the cross point of the diagonals, and horizontal beams of "L" shaped cross section at the top and bottom of every panel. Each leg had a number of gussets welded at the top, middle and bottom for each panel; such that the described members could be bolted on using fluted shank bolts. Other wires, cables and antennae were attached as needed for operation. Within each section, the panels were numbered 1, 2 and 3, with 1 at the bottom and 3 at the top. (This is opposite to the direction convention used to number the sections.)

An independent engineering firm determined that this tower needed structural reinforcement in order to safely accept new equipment installation. These changes were to be made on sections 10 and 47. At the time of the accident, two men were on the tower changing the diagonal trusses. Several other support personnel were on the ground. One employee was at the junction of sections 47 and 48. It is unknown where the other employee was on the tower. The tower buckled at section 47 and fell. The bottom section fell nearly straight over. The top section fell with section 47 impacting the ground. The amount of damage to section 47 precluded preservation of that section. As a result, this examination centers on the attachment points of diagonals extending from section 47 to section 48. Gross examination of section 10 indicates that all new members and bolts were in place. New members were distinguished from the old members as the old members were painted, while the new ones had only a primer gray coat. Because section 10 was apparently intact, and did not fail, OSHA performed no further analysis on that section.

In order to identify the parts, a system of notation was adopted as shown in Figure 1. The identification of the upright legs as "a", "b", and "c" is equivalent to that used by OSHA elsewhere. A face between leg a, and leg b is noted as: $\langle a-b \rangle$. Other faces are noted similarly. Gussets are named as shown in Figure 1. The holes in the analyzed gussets are named as shown in Figure 2. In order to provide correlation with any other analysis, it was noted at the end of the examination that two of the legs were identified as 48 NE and 48W. We did not note the identification of the third leg. Leg 48 NE is the same as our designation "a". Leg 48 N is the same as our designation "b".



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Figure 2: Hole naming conventions for gussets

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Data from Table 1 shows that the actual deformation to be 2.48mm (0.098"). All of the other holes showed unremarkable eccentricity.

Also, as shown in photo 4, the holes had markings consistent with the flutes of the connecting bolts.

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Photo 4: Hole H2, <a-c>, leg a, showing marks from bolt shank flutes

The data in table 1 shows all the holes to have deformations typically under 9% relative to through-hole measurements, with the exception of holes T1 and T2 of top gusset <a-b>, leg b. The biggest deformations appear to be in the tensile direction of each member. Some deformation might be expected from normal installation and wear. The amount of deformation shown in these two holes is much more than in any other and appears new. The shanks of the other holes still have paint in place and other marks do not appear to be new.

The conclusion of observations and measurements is that of the measured holes and other observed holes, only T1 and T2 of the top gusset of $\langle a-b \rangle$, leg b had bolts installed at the time of the accident.

Holes in gussets on the top ends of legs a, and b were examined as well as the brackets on the diagonal trusses at the center where they would have been attached to the redundant. Holes in the end gussets were measured and recorded. Photo 1 shows the location of the top gusset $\langle a-b \rangle$, Leg a. Each hole was measured with a digital Mitutoyo caliper. The holes were measured in three directions. Each hole was measured in the direction parallel to the leg direction, parallel to the tension axis of the member, and perpendicular to the tension direction of the member. Three measurements were made in each direction. The first was a through-hole measurement, inserting the caliper completely through the hole. The second measurement was the top side of the gusset as it lay on the floor of the warehouse. The third measurement was made on the opposite face of the gusset where the truss was attached. A total of nine measurements were made for each hole. These measurements and some rudimentary analysis are in Table 1. (Table is at the end of the report.)



Photo 1: Leg a showing top gusset between leg a, and leg b. Note the horizontal element still in place. Bolts were removed prior to fall.

Visual examination of these holes showed that only the holes T1 and T2 of the top gusset $\langle a-b \rangle$, leg b, were visibly deformed. The other four holes did not show any remarkable deformation. Supplied information indicated that the bolts in holes T1 and T2 of the top gusset of $\langle a-b \rangle$, leg a, had been removed before the fall. They thus, provide an internal control for the appearance and eccentricity of the holes. Holes H1 and H2 of gusset $\langle a-c \rangle$, a, were also reported to be removed before the fall. The appearance and measurements of these holes are consistent with T1 and T2, $\langle a-b \rangle$, leg a.

Photo 2 shows hole T2 of top gusset $\langle a-b \rangle$, leg b. The scale in the photo indicates a diagonal deformation. Photo 3 is a similar photo of hole T2 of top gusset $\langle a-b \rangle$, Leg a. Little notable eccentricity is visible.

	Frac Lev	0.0184	0.0098	0.0526								0.0219	0.0397	0.0522							0.0333	0.0320	0.0400					
	Std DeV	0.0124	0.0069	0,0366.								0.0148	0.0290	0.0363							0.0225	0.0221	0.0279					
	Mean	0.6758	0.7076	0,6956						·		0.6763	0.7294	0.6958							0.6763	0.6916	0.6977					
	2120 21-2	0.6630	0.7095	0.6745	0.6823	0.0242	0.0355	7.01%		1./3%		0.6660	0.7640	0.6685	0.6995	0.0799	14.71%	10000	0.30%		0.6670	0.6705	0.6945	0.6773	0.0221	0.52%	4.12%	
- -	<a-b>, b, 11 💽</a-b>	0.6735	0.7170	0.6760	0.6888	0.0244	0.0355	6.46%		0.37%		0.6730	0.7635	0.6975	0.7113	0.0658	13.45%	10100	3.04%		0.6610	0.6630	0.6920	0.6720	0.0258	0.30%	4.69%	
	a-c>;a H2	0.6930	0.7020	0.7475	0.7142	0.0292	0.0409	1.30%		7.86%		0.6890	0.7025	0.7310	0.7075	0 0303	1.96%		6.10%		0.7085	0.7185	0.7200	0.7157	0.0087	1.41%	1 62%	
TABLE 1	<a-c>, a, H1 💦</a-c>	0.6895	0.7015	0.7370	0.7093	0.0247	0.0348	1.74%		6.89%		0.6985	0.7115	0.7475	0.7192	0.0353	1.86%		7.02%		0.7015	0.7115	0.7400	0.7177	0.0278	1.43%	5 49%	
	<a-b> a, T2</a-b>	0.6680	0.7015	0.6745	0.6813	0.0178	0.0261	5.01%		0.97%		0.6585	0.7015	0.6595	0.6732	0.0365	6.53%		0.15%		0.6570	0.6985	0.6695	0.6750	0.0315	6.32%	1 90%	
.	<a-b>, a, T1 🛛</a-b>	0.6680	0.7140	0.6640	0.6820	0.0278	0.0407	6.89%		~0.60%		0.6730	0.7335	0.6710	0.6925	0.0513	8.99%		-0.30%		0.6630	0.6875	0.6700	0.6735	0.0187	3.70%	1.06%	
	Hole # face, leg hole)		And the second se	A Contract Contract Bottom	Mean	Sig Deviation	Eracitonal deviation (Std Dev) Wean	Percent hole deformation (Top	relative to through-hole measurement	Fercent note deformation (Eottom		Tensile Diagonal Through Hole	Tensile Diagonal Top	Tensie Diagonal Bottom	Mean	Elactional deviation (Std: Dev) Mean	Percent hole deformation (Top	relative to through-hole measurement	Percent hole: deformation (Bottom relative to through hole measurement		Cross Tensile Diagonal Through Hole	Cross Tensile Diagonal Top	Cross Tensile Diagonal Bottom	A CONTRACT OF	 Fractional deviation (Std Dev) Mean 	Percen hole deformation (Top	relative to intolognatiole measurement Percant Internation (Bottom	relative to through-hole measurement

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Numeric values are in inches <a-b> means the face between legs a and b, etc.

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Figure 1: Naming conventions



Figure 2: Hole naming conventions for gussets

Structural Analysis

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September 24, 2002

Collapse of KDUH-TV

Antenna Tower

by J. Hugh Bradburn



Contents

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Analysis - Proposed Upgrade Conditions

Opinions

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Basis and Reasons for Opinions

Reasoning and Methodology Structural Analysis Strength Evaluation

Cases Testified

Exhibits

- 1. Description of files produced by the respective structural analyses.
- 2. Data plots for structural analyses.
- 3. Results for leg strength calculations.
- 4. Description of computer results to support leg strength calculations.
- 5. Vitae
- 6. Floppy disk containing all computer generated files

Background

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In November of 2002 I was contacted by Mohammad Ayub, Chief Structural Engineer for OSHA, inquiring about my possible interest in investigating the collapse of the KDUH-TV communication tower that occurred September 24 near Alliance, Nebraska. I told him that I would be interested and we then discussed some of the details concerning the collapse. We both noted the similarities between the collapse of this tower and the collapse of a similar tower in Jackson, Mississippi in October of 1997 which I also investigated. I was later contacted by Glenn Taylor of OSHA - Region VII to finalize the agreement. It was agreed that I would perform two structural analyses of the tower and report to OSHA the results of these analyses, including any opinions that I might draw from these analyses. One analysis would be concerned with the tower as it existed at the time of collapse and the second analysis would be concerned with determining the adequacy of proposed upgrades to the tower to carry loading additional to the original design loads.

These structural analyses were subsequently performed using computer programs developed by myself over the past twenty five years. The validity of these computer programs has been verified through many years of use by nationally reputable engineering firms involved in the analysis and design of cable supported tower structures. The necessary data and information, required to create a computer model of this tower, was provided primarily by OSHA. This data and information included; (1) design drawings dated 1977, 1993, and 2002, (2) a report "Rigorous Computer Structural Analysis and Evaluation of the KDUH-TV 1965 Ft. Guyed Tower Hemingford, Nebraska" prepared by Structural Systems Technology, Inc. in May of 1999, (3) miscellaneous photographs and newspaper articles concerning the collapse, (4) weather data for Alliance, Nebraska for September 23rd and 24th, 2002, (5) miscellaneous information requested from OSHA by myself and transmitted by email and fax, (6) information transmitted verbally by Mohammed Ayub concerning conditions that existed at the time of the collapse.

Analysis results are produced for three wind directions where; Direction 1 is normal to a tower face, Direction 2 is 180 degrees from Direction 1, and Direction 3 is parallel to a tower face.

Note, the design drawings number tower sections from top to bottom, whereas, the computer program used employs a numbering system which numbers the tower sections from bottom to top. The section numbers referred to in the body of this report are those of the design drawings

Analysis - At Time of Collapse

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Reports indicate that workers were performing modifications to the tower at two locations on the tower at the time of collapse, one at section 10 (approximately 1610 feet) and the other at section 47 (approximately 490 feet), as directed by the proposed upgrade. Wind data indicates that the wind speed at time of collapse was approximately 8 miles per hour. This wind speed has negligible effect on the analysis results and was neglected in the analysis.

The results of this analysis^{*} indicate that the maximum load being carried by a tower leg at section 10 was approximately 90 kips, well below the minimum strength range^{**} of the legs which is 137 kips to 150 kips.

The maximum load being carried by a leg at section 47 was approximately 260 kips. An evaluation of the leg strength indicates that a tower leg, at this level, could carry a load of approximately 968*** kips under normal circumstances. This value assumes a critical buckling length of 5 feet. However, if the redundant member bracing the leg at mid height of a panel were to be removed/disconnected at this location in the tower the load carrying capacity of a leg would be reduced to approximately 692*** kips per leg. Furthermore, if both the redundant and a diagonal were to be removed/disconnected at this location the load carrying capacity of a leg would be reduced to a range** of approximately 260 to 285 kips per leg or less. The value of 260 kips is deemed to be close to the true value and the value of 285 kips represents an upper limit.

One may conclude from this analysis that if a redundant member and a diagonal member were to be removed/disconnected, at section 47, that buckling of the tower legs would be possible. If the tower legs were to buckle the collapse of the entire tower would be inevitable.

- * These analysis results are contained in text files C_0.raw, and C_0.bf which are described in exhibit 1.
- ** A range for strength is given since the strength values given depend on the torsional stiffness of the tower above the critical section which must be approximated. Strength calculations are presented in exhibit 3.

*** The calculation of this value is described in exhibit 3.

Analysis – Proposed Upgrade Conditions

A proposed upgrade of the tower is presented in the report "Rigorous Computer Structural Analysis and Evaluation of the KDUH-TV 1965 Ft. Guyed Tower Hemingford, Nebraska" prepared by Structural Systems Technology, Inc. in May of 1999.

An analysis of this proposed configuration was performed according to the EIA Standard RS-222-C for a uniform wind pressure of 65 pounds per square foot with no ice, which is the applicable TIA/EIA standard for this tower. The results of this analysis* indicate that the proposed design meets all requirements of EIA Standard RS-222-C. Although the results are not presented analysis indicates that the proposed configuration would not meet the more stringent EIA Standard RS-222-F standard.

* These analysis results are contained in text files C_65.raw, C_65.des and C_65.bf which are described in exhibit 1. Corresponding data plots are contained in exhibit 2.

Opinions

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- 1. This "tower structure" was composed of legs, struts, diagonals and redundant leg braces.
- 2. The concept, philosophy, and use of "redundant" members to brace another member are common to all structures and well known to structural engineers.
- 3. Each component (member) of the "structure" was crucial to the structural integrity of the entire tower.
- 4. The removal of any member without proper bracing or substitution could result in the collapse of the entire tower.
- 5. Each member of the structure is subject to variation of stress due to factors such as; construction, wind, temperature, settlement, adjustment of guy tension and removal of members.
- 6. Industry standards governing the analysis and design of tower structures account for the aforesaid variations.
- 7. Over time tower structures will require maintenance and renovation and are therefore designed to facilitate such maintenance and renovation. In order for such maintenance and renovation to be performed, the foregoing standards must be adhered to.
- 8. It would be very difficult, if not impossible, to remove a diagonal member in this structure without disconnecting the clamping mechanism at the intersection of the redundant and diagonal members.

- 9. The KDUH tower structure in Alliance, Nebraska collapsed in September of 2002 due to the removal/disconnection of a redundant member and a diagonal member without proper bracing or substitution for these members.
- 10. Dealing with a guyed tower structure of any design is a risky and dangerous task regardless of whether it involves analysis, design, construction or maintenance. It is the responsibility of anyone who chooses to deal with such a structure, especially for pay, to understand completely the structure itself as well as the associated risks and dangers.
- 11. It is common practice throughout the industry, be it analysis, design, construction or maintenance to brace or substitute for members while they are being replaced.

Basis and Reasons for Opinions

- 1. Education, experience, and background as a structural engineer.
- 2. Same as 1.

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- 3. Same as 1.
- 4. Same as 1.
- 5. Same as 1.
- 6. Knowledge and experience in using the AISC "Manual of Steel Construction" and the TIA/EIA-222-C "Structural Standards for Steel Antenna Towers and Antenna Supporting Structures."
- 7. Same as 6.
- 8. Because this connection consists of "lugs" attached to diagonals and redundant and bolted with two bolts such that both bolts must be removed at the same time in order to remove a diagonal.
- 9. The results of the analysis presented earlier indicated that if a redundant and a diagonal are removed/disconnected in the same panel and same face at the same time that buckling of a leg is possible. Information provided by OSHA via Mohammed Ayub, based on inspection of the collapsed tower, indicated that in two faces of the bottom panel of section 47 (as per original design drawings) a diagonal had been removed and that the redundant was disconnected at the lugs. Since the calculated load in a leg in section 47 was approximately the same as the strength of the leg it is highly probable that failure of the tower was initiated here. In addition, the provided information indicates that the main strut at the top of section 48 was also disconnected in one of these faces, which would cause the leg strength to be reduced even further.
- 10. Experience and knowledge gained by developing computer programs for this industry and working with experienced people in the industry who are responsible for designing and building all kinds of tower structures. In addition I have been involved in the post-mortem analysis of other such

tower failures, most notable the 1996 failure of the Cedar Hill, Texas tower and the 1997 failure of the Jackson, Mississippi tower.

11. Through discussions with reputable people in the guyed tower industry. Most notably engineers at Kline Towers, and Anderson-Foreman Engineers, of Columbia, South Carolina and 4SE Engineers, Inc. of Charleston, South Carolina.

Reasoning and Methodology

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Structural Analysis

The computer program used to analyze the structure for loads is based on the standard methodology of stiffness method analysis and accounts for large displacements using a standard iterative approach. The structural model used by the program to model the tower is now officially recognized by the TIA/EIA (1). The cable element used by the program to model the guys is an exact cable stiffness element developed by myself and tested against a similar element (2) published in the literature. A non standard feature of the program is its' ability to reduce compression in long slender diagonal members and to this point has had no peers to test against.

As is the case with all computer programs, the quality of the results of a program can be no better that the quality of the data entered into the program to produce the results. In this report, the results produced are felt to be accurate to within plus or minus 5% of the true behavior of the structure.

(1) TIA/EIA-222-G "Structural Standard for Antenna Supporting Structures and Antennas" Section 3.4.

(2) Peyrot, A. H. and Goulois, A. M., "Analysis of Cable Structures", Computers and Structures, 1979, vol. 10, pp 805-813.

Strength Evaluation

Three sources of methodology were used to evaluate the strength of members in this report; AISC Manual (1), (2), Mathcad computer programs, and buckling models conceived and evaluated by myself.

The AISC Manual (1) has been and is currently the standard specified by TIA/EIA for evaluating the allowable strength for all tower members and

was the basis for evaluating the adequacy of the proposed upgrade to the tower.

The Mathcad computer programs were developed by myself over the past ten years and are the basis for evaluating the strength of the tower legs at the time of failure. These programs are based on standard elastic buckling theory used by structural engineers for more than a hundred years and the stiffness method of structural analysis which has been the standard for structural analysis for at least forty years. These programs make it possible to create and analyze reasonably sophisticated buckling models in an efficient manner. These programs have been tested against a wide range of available results and found to be correct and accurate for standard models.

Unfortunately, the ability to predict buckling strength depends on ones ability to conceive, create, and analyze the "critical buckling mode" of a given structure for which there may be many possibilities. The critical buckling mode is defined as that mode that produces the minimum buckling strength. In many cases it is obvious what the critical buckling mode should be, however, as is the case with this structure, the concept of the critical buckling mode may not be obvious to all. Although no test or peer review of this mode is available, it is my opinion that a true expert in this field would have no difficulty in accepting the buckling mode presented in this report as the critical mode which ultimately caused the tower to collapse.

It is my opinion that the strength predicted by these Mathcad computer programs for the assumed critical buckling mode is within plus or minus 10% of the true strength since some small approximation is required to obtain the results.

- (1) AISC "Manual of Steel Construction Allowable Stress Design", Ninth Edition
- (2) AISC "Manual of Steel Construction Load and Resistance Factor Design", Third Edition.

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I gave deposition in OSHA vs. LeBlanc & Royal Telecom, Inc in December of 1998.

I gave deposition in WLBT-TV vs. LeBlanc & Royal Telecom, Inc in May of 1999, Case No. 3:98CV703 BN, US District Court for the Southern District of Mississippi, Jackson, MS.

Exhibits

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Exhibit 1 Description of Analysis Results*

C_O.raw File that contains the gross results of the tower analysis for conditions that existed at the time of collapse

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- C_0.bf File that contains all bar forces for all members for all wind directions for conditions that existed at the time of collapse
- C_65.raw File that contains the gross results of the tower analysis for proposed configuration
- C_65.des File the compares maximum member force with allowable member force for all members (Based on AISC/ASD)
- C_65.bf File that contains all bar forces for all members for all wind directions for proposed configuration

* All files contained on floppy disk, Exhibit 6, and stored in MSDOS text format

Exhibit 2

Data Plots for Tower Analyses

Notation

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Wind Direction 1	->	y Direction
Wind Direction 2	->	-y Direction
Wind Direction 3	->	-x Direction
Units	->	Feet and Kips
Dx	->	Displacement in x direction
Dy	->	Displacement in y direction
Vx	->	Shear in x direction
Vy	->	Shear in y direction
M×	->	Bending Moment about × a×is
My	->	Bending Moment about y axis

Wind Direction - 1



1965' DRESSER TOWER(EIA-222-C, 65PSF) HEMINGFORD, NE, KDUH.DAT 19-NOV-02

Wind Direction - 1

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Wind Direction - 1

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Wind Direction - 2

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1965' DRESSER TOWER(EIA-222-C, 65PSF) HEMINGFORD, NE, KDUH.DAT 19-NOV-02

Wind Direction - 2

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Wind Direction - 2

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Wind Direction - 3



1965' DRESSER TOWER(EIA-222-C, 65PSF) HEMINGFORD, NE, KDUH.DAT 19-NOV-02

Wind Direction - 3

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1965' DRESSER TOWER(EIA-222-C, 65PSF) HEMINGFORD, NE, KDUH.DAT 19-NOV-02

Wind Direction - 3

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Exhibit 3

Leg Strength Calculations* (Buckling Strength)

A. Buckling Model

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- B. Sample Results and Summary of Strength Results for Section 47
- C. Sample Results and Summary of Strength Results for Section 10
- D. Calculation of Leg Strength for 5' Effective Buckling Length
- E. Calculation of Leg Strength for Non Torsion/Sway Mode
- Computer results shown were obtained from fairly large proprietary Mathcad program which is capable of determining buckling loads for column like structures and developed by myself

Reference:C:\w\Sap_mcd\Bsap.mcd

$$xy := \begin{pmatrix} 0 & 0 \\ 0 & 60 \\ 0 & 120 \\ 0 & 180 \\ 0 & 240 \\ 0 & 300 \\ 0 & 360 \\ 0 & 420 \\ 0 & 420 \\ 0 & 420 \\ 0 & 420 \\ 0 & 420 \\ 0 & 420 \\ 0 & 420 \\ 0 & 540 \\ 0 & 540 \\ 0 & 560 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 360 \\ 10 & 420 \\ 10 & 500 \\ 10 & 420 \\ 10 & 500 \\ 10 & 420 \\ 10 & 500 \\ 10 & 540 \\ 10 & 540 \\ 10 & 540 \\ 10 & 540 \\ 10 & 540 \\ 10 & 540 \\ 10 & 540 \\ 10 & 540 \\ 10 & 540 \\ 10 & 540 \\ 10 & 540 \\ 10 & 540 \\ 10 & 540 \\ 10 & 540 \\ 10 & 540 \\ 10 & 540 \\ 10 & 560 \\ 11 & 16 \end{pmatrix}$$

$$axial := \begin{pmatrix} 7 & 12 \\ 8 & 13 \\ 9 & 14 \\ 10 & 15 \\ 11 & 16 \end{pmatrix}$$

$$a := \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix}$$

$$a := \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix}$$

$$a := \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix}$$

$$a := \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix}$$

$$b = \begin{pmatrix} 1 \\ 2 \\ 13 \\ 14 \\ 14 \\ 15 \\ 15 \\ 16 \end{pmatrix}$$

$$b = \begin{pmatrix} 1 \\ 2 \\ 13 \\ 14 \\ 14 \\ 15 \\ 15 \\ 16 \end{pmatrix}$$

$$b = \begin{pmatrix} 7 & .442 \\ 0 & .0 \end{pmatrix}$$

$$JL(P) := \begin{pmatrix} .1 & 0 & 0 & 6 \\ 0 & -P & 0 & 11 \end{pmatrix}$$

$$Iter := 3 \qquad \alpha := 0 \qquad d_{0,0} := 0$$

$$i := 0, 1..20$$

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$$P_i := 250 + 1 \cdot i$$

 $\Delta_{i} := frame (A, a, E, I, xy, axial, beam, lnk, s, r, d, JL(P_{i}), bl, Iter, \alpha)_{I}$



Detection of Critical Buckling Load

((212)	
	223	
1	234	
	244	
	255	
Pcr :=	265	
	277	
1	288	
	298	
	309	
	(320))

k_j := .1 · j

j := 0..10

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Plot of Buckling Load vs Torsional Restraint Sriffness

 \blacksquare Reference:C:\w\Sap_mcd\Bsap.mcd

$$i := 0, 1..20$$

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$$P_i := 130 + 1 \cdot i$$

 $\Delta_{i} \coloneqq frame \left(A, a, E, I, xy, axial, beam, lnk, s, r, d, JL \left(P_{i}\right), bl, Iter, \alpha\right)_{l}$



Detection of Critical Buckling Load

	(114)	
	125	
	137	
	148	
	158	
Pcr :=	169	
	180	
	190	
	201	
	212	
	(223)	

k_{.j}:= .1∙j

j := 0.. 10

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Buckling Load Summary



Plot of Buckling Load vs Torsional Restraint Sriffness

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d := 4.375
$$r := \frac{d}{4}$$
 $A := \frac{\pi \cdot d^2}{4}$ $I := A \cdot r^2$ $r = 1.094$
 $A = 15.033$

Pcr :=
$$\frac{\pi^2 \cdot E \cdot I}{KL^2}$$
 Pcr = 1429.807 Elastic Buckling Theory Py := A \cdot Fy
Py = 1503.3

$$f(\lambda) := \begin{cases} \Pr \leftarrow \frac{.877 \cdot Fy}{\lambda^2} \cdot A & \text{if } \lambda > 1.5 \\ Pcr \leftarrow .658^{(\lambda^2)} \cdot Fy \cdot A & \text{if } \lambda \le 1.5 \\ Pcr \end{cases}$$

$$\lambda := \frac{60}{\pi \cdot r} \cdot \sqrt{\frac{Fy}{E}} \qquad Pcr := f(\lambda) \qquad Pcr = 968.118 \qquad \text{Inelastic Buckling - LRFD}$$



 $\operatorname{lam}_{i} := \frac{L_{i}}{\pi \cdot r} \cdot \sqrt{\frac{Fy}{E}}$ $P2_{i} := f(\operatorname{lam}_{i})$

i := 1..200

 $L_i := 1 \cdot i$

 $P1_{i} := \frac{\pi^{2} \cdot E \cdot I}{\left(L_{i}\right)^{2}}$

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Э D + Reference:C:\w\Sap_mcd\Bsap.mcd

$$xy := \begin{pmatrix} 0 & 0 \\ 0 & 60 \\ 0 & 120 \\ 0 & 180 \\ 0 & 240 \\ 0 & 300 \\ 0 & 360 \end{pmatrix} beam := \begin{pmatrix} 1 & 2 & 1 \\ 2 & 3 & 1 \\ 3 & 4 & 1 \\ 4 & 5 & 1 \\ 5 & 6 & 1 \\ 6 & 7 & 1 \end{pmatrix} A := \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix} \begin{pmatrix} \pi \cdot \rho^2 \end{pmatrix} I := \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix} \begin{pmatrix} \pi \cdot \rho^4 \\ 64 \end{pmatrix} r := \begin{pmatrix} 1 & 0 & 0 & 0 & 0 \\ 2 & 0 & 1 & 1 & 0 \\ 3 & 0 & 1 & 1 & 0 \\ 5 & 0 & 1 & 1 & 0 \\ 6 & 0 & 1 & 1 & 0 \\ 6 & 0 & 1 & 1 & 0 \\ 7 & 0 & 1 & 0 & 0 \end{pmatrix}$$

 $axial_{0,0} := 0$ $a_0 := 0$ $lnk_0 := 0$ $d_{0,0} := 0$ $s_{0,0} := 0$ $bl_{0,0} := 0$

 $JL(P) := \begin{pmatrix} .1 & 0 & 0 & 4 \\ 0 & -P & 0 & 7 \end{pmatrix}$ Iter := 3 α := 1

i := 0,1..20

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 $P_i := 805 + i$

 $\Delta_i := \text{frame}(A, a, E, I, xy, axial, beam, lnk, s, r, d, JL(P_i), bl, Iter, \alpha)_1$

frame is a function that returns a vector with four subvectors. The first is the sub-zero vector and represents the member forces, the second is the sub-one vector and represents the displacements, the third is the sub-two vector and represents the spring forces, and the fourth is the sub-three vector and represents the nodal equilibrium check.

Member Forces are given in the F matrix which contains the local coordinate forces for all beams, followed by all axial elements. Output is in the following order for each element, Pa, Pb, Va, Ma, Vb, Mb

∆ represents the global deflections and is ordered rows as nodes and columns as x,y, & rz.

The spring forces are given in the S matrix for each spring.

 ΔP contains the summation of forces at each joint and is ordered rows as nodes and columns as ΣFx , ΣFy , & ΣMz .



Fy := 100 E := 29000 $\phi := 4.375$

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 $r := \frac{\phi}{4}$ $A := \frac{\pi \cdot \phi^2}{4}$ $I := A \cdot r^2$ $Py := A \cdot Fy$ A = 15.033 r = 1.094 I = 17.984 Py = 1503.301 $.39 \cdot Py = 586.287$

KL := 120 Pcr := 811 Elastic Buckling from Mathcad Given $Pcr = \frac{\pi^2 E \cdot I}{KL^2}$ ans := Find(KL) ans = 79.667

$$f(\lambda) := \begin{vmatrix} \Pr \leftarrow \frac{.877 \cdot Fy}{\lambda^2} \cdot A & \text{if } \lambda > 1.5 \\ \Pr \leftarrow .658^{(\lambda^2)} \cdot Fy \cdot A & \text{if } \lambda \le 1.5 \\ \Pr \\ r \\ \end{cases}$$

KL := 79.667
$$\lambda := \frac{KL}{r \cdot \pi} \cdot \sqrt{\frac{Fy}{E}}$$
 Pcr := f(λ)

Pcr = 691.997 Approximate Inelastic Buckling - LRFD

Exhibit 4

Support Calculations for Leg Strength Calculations

A. Definition of Equivalent Torsional Stiffness

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B. Hand Calculation of Equivalent Torsional Stiffness

C. Description of Computer Generated Support Files*

* All files contained on floppy disk, Exhibit 6, and stored in MSDOS text format

Description of Computer Generated Support Files

k_47.dat

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The tower model is modified so that the tower above Section 47, Panel 1 may rotate freely with respect to the tower below. The only torsional restraint is then provided by the guys above. A moment of one foot kip is then applied at the bottom of the upper part of the tower producing the results contained in this file

k_10.dat The tower model is modified so that the tower above Section 10, Panel 1 may rotate freely with respect to the tower below. The only torsional restraint is then provided by the guys above. A moment of one foot kip is then applied at the bottom of the upper part of the tower producing the results contained in this file

APPENDIX C (TOTAL SHEETS 2)

GENERAL INSTRUCTIONS

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STRUCTURAL SYSTEMS TECHNOLOGY, INC.

GENERAL INSTRUCTIONS

TENSION DIAGONAL REPLACEMENT - REDUNDANT HORIZONTAL INSTALLATION

1. Tension Disgonal Member Replacement:

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- Prior to removal of the existing diagonal, a temporary system with edequate tension capacity shall be installed parallel to the existing member. The temporary system shall consist of two adequately sized (minimum %" diameter) chokers tied around the towar legs, one above the top gusset plate and one below the bottom gusset plate, and a 3 ton Come-A-Long hooked securely to the eyes of the chokers at each end. The temporary system is then tightened using the Come-A-Long. The temporary system shall be tightened only enough to allow the removal of the existing diagonal. DO NOT OVER-TENSION THE TEMPORARY SYSTEM.
- Once the temporary system is tightened, carefully remove the existing diagonal member and immediately
 replace it with the new diagonal member and install and tighten the bolts.
- Remove and replace only one diagonal member at a time.
- In towers with redundant horizontal members incorporated and installed in the original design and tower installation, when replacing the diagonal members, DO NOT REMOVE THE REDUNDANT HORIZONTAL MEMBERS.

2. <u>Redundant Horizontal Installation:</u>

- For "Tension Only" X-Bracing systems with rod diagonals, install the center plate, at the point where the diagonal rods cross each other, and tighten the U-Bolts.
- Install new Collars with gusset plates, on the tower legs, at the center of the panel, at the same level with the center plate.
- Install the redundant horizontal members between the collars and the center plate, making sure that the redundant horizontal members are leveled properly.
- Install the redundant horizontal members one face at a time.
- In panels with back-to-back angle X-Bracing, one diagonal member is in full length and the other one is in two pieces, which are bolted to a center plate at the crossing point of the diagonals. The center plate is also bolted to the full-length diagonal member. Installation of redundant horizontal members in panels with this type of bracing system will require the removal of the two-piece diagonal member and replacement of the center plate. To safely perform this procedure, A TEMPORABY SYSTEM, AS OUTLINED IN ITEM 1. MUST BE INCORPORATED.

P.O. BOX 876 - 6867 ELM STREET, SUITE 20D, MCLEAN, VIRGINIA 2210-38851 - TELEPHONE: 800.887.6555 OR 703.358.9765 - FAX: 703.448.0979