ANALYSIS OF RADIO COMMUNICATION TOWERS SUBJECTED TO WIND, ICE AND SEISMIC LOADINGS

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Master of Science

By

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The undersigned, appointed by the Dean of the Graduate School, have examined the thesis entitled

ANALYSIS OF RADIO COMMUNICATION TOWERS

SUBJECTED TO

WIND, ICE AND SEISMIC LOADINGS

Presented by Eric J. Sullins

A candidate for the degree of Masters of Science in Civil and Environmental Engineering

And hereby certify that in their opinion it is worthy of acceptance.

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ANALYSIS OF RADIO COMMUNICATION TOWERS SUBJECTED TO WIND, ICE AND SEISMIC LOADINGS

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ABSTRACT

The Missouri Department of Transportation radio communication tower network is currently out of date with respect to current tower building codes. The network was created in the 1950s and 1906s as part of the U.S. military civil defense system. The network was designed using the TIA-222-C (1976) or earlier. Today the current code in practice is the TIA-222-F (1996) code. There is a need to assess the condition of the towers in the network and also to determine if they are up to date with current code. A condition indexing (CI) system is a reliable way to assess this problem. However, an analytical method of determining the input parameters for the CI needs to be determined. Therefore the objective of this research is to develop a systematic evaluation and assessment method that could provide the necessary information for the repair and maintenance of the tower network.

Two towers were selected for this project to act as model towers. One tower is guyed, the Taum Sauk tower, and the other is freestanding, Kansas City tower. Both towers are analyzed using the TIA-222-F for wind and ice loadings. The Taum Sauk tower is then analyzed for seismic loading. Also a parametric study to determine the effects of deterioration of tower components on the tower as a whole is completed on the Taum Sauk tower.

The controlling components of the Kansas city tower were found to be the diagonal bracings. The critical bracings were found to be at 88.9% of their maximum

capacity. The maximum capacities of the other components of the tower were found not to exceed 51.1% capacity. Therefore, the tower passes for the current code.

The parametric study was conducted on the Taum Sauk tower under wind, ice and seismic loadings. For the wind and ice analysis it was determined that the bracing on the tower controls the structural integrity of the tower. If the braces are damaged by as little as 10%, the capacity exceeds 100% and the tower fails. When damage is introduced to the guys and the legs, the tower fails at 25% and 42% damage, respectively.

The parametric study completed using seismic loading did not provide information about failure. The results of the parametric study showed the increase or decrease in axial force of the components due to deterioration in one or all of the tower components. It is seen that there is a 0.5 kip increase in axial force in the legs when the guy cables are damaged, and a 0.25 kip increase in the legs when the whole tower is damaged to 50% original cross sections. A 0.15 kip increase in axial force in the guys is seen when the legs are damaged to 50% their original cross sections. Damage to either the tower legs or the guys can cause significant increases in axial forces when subjected to seismic loadings.

In this project the towers were analyzed under wind, ice, and seismic loading and the results indicate that some components of the towers are critical and could control the failure. It is recommended that detailed inspection of the towers' critical components be performed to perform a detailed risk assessment.

1 Introduction

1.1 Statement of the Problem

Missouri Department of Transportation's radio communication tower system was developed in the 1950s and 1960s as part of a military civil defense system. The tower system is now utilized by the Department of Transportation along with highway patrol, fire and other emergency agencies as a communication system. Below is a figure picturing the entire tower system.

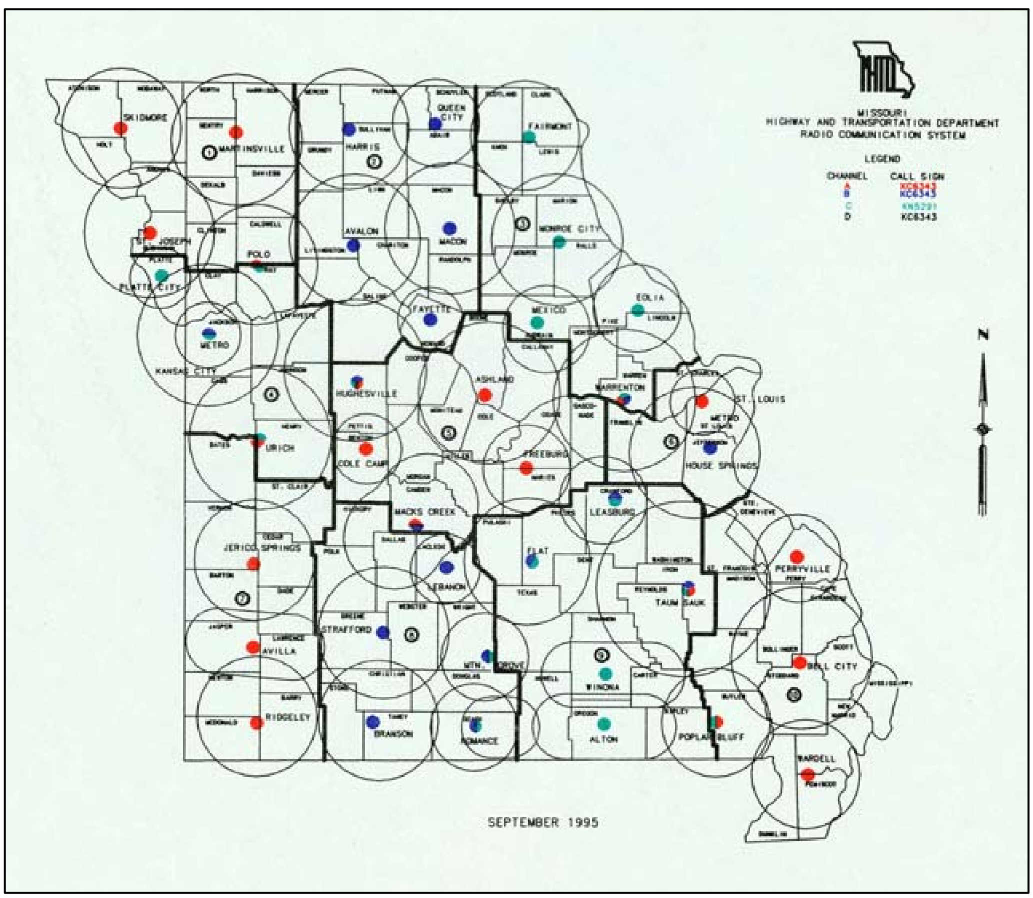


Figure 1.1. MODOT radio communication tower system

The tower system is now around 50 years in age, and was developed using the TIA-222-C (1976) tower building code. This code is now out of date, as the current code is the TIA-222-F (1996) version with the G-version in draft form. Therefore the tower system may not be up to current code standards. Along with the changes in the code, the tower system has been subjected to 50 years of natural hazards causing deterioration in the structures. The major issue is how the tower network will behave in times of emergency, including floods, severe storms and earthquakes. “The limitations of that aging system were the chief obstacle encountered during [MoDOT’s] first-ever statewide earthquake drill in May [2002]. Field crews trying to solve the problems of a mock 6.7-magnitude quake in eastern Missouri sometimes were unable to communicate with their headquarters or each other” (St Louis Post Dispatch, August 2002).

Wind and ice loads are main contributors of load on a tower. However earthquake loading could now be an important factor in southern Missouri. The New Madrid fault line poses a threat of earthquake activity that makes a detailed dynamic analysis of the tower system necessary.

Another problem the Missouri Department of Transportation is facing is the upkeep of the aging towers along with staying within a budget that is decreasing. To deal with the decrease in funding, a systematic evaluation of the tower systems performance under varying loadings is being developed. This is a condition indexing system that includes assessment of the conditions of each part of the tower structure and the foundation.

1.2 Objective

The objective of this thesis is to develop a systematic evaluation and assessment method that could provide necessary information for the repair and maintenance of the tower network, and the development of a condition indexing system. This system will organize the towers by importance in the network along with the condition of the tower. To be able to rank the towers with regard to structural integrity, it is important to develop a procedure for modeling and analyzing the towers using the latest codes and also under earthquake loadings. The models of the towers will be loaded with wind and ice to evaluate their performance using the new code requirements. The models will also be used to evaluate the effect of deterioration of the tower components on the overall behavior of the tower structure and its failure characteristics.

Therefore the objective of this thesis is to build structural dynamic models for two typical towers of the Missouri telecommunication tower network. The two towers selected for this study are the Taum Sauk tower located on Taum Sauk Mountain in Southeastern Missouri and the Kansas City tower. The wind and ice loads on each tower will be modeled using ERITower, while seismic analysis of Taum Sauk will be performed using SAP2000. Along with the study of the original structures, a parametric study will be performed on the Taum Sauk tower, using both modeling tools, to show the effect of member deterioration on the structural integrity of towers.

The specific tasks of this project are:

- Review past studies pertaining to dynamic analysis of towers.
- Verify modeling tools are accurate using a simplified model.
- Model Kansas City and Taum Sauk towers using ERITower and SAP softwares.

- Analyze towers using TIA222-F standards for wind and ice loads.
- Analyze Taum Sauk tower dynamically using response spectrum and time history modal analyses.
- Conduct a parametric study for 10, 20, 30, 40 and 50 percent damage to members of Taum Sauk tower.

Once these objectives have been attained, the data can be used to help create an accurate condition indexing system for the tower network, which was the objective of Tulasi (2005).

1.3 Scope of Research

Wind, ice and earthquake loads are some of the major static and dynamic loads seen on communication towers. A literature review of how towers are analyzed for dynamic loads is given in chapter 2. ERITower v.9 will be used to model both the Kansas City and Taum Sauk towers. Chapter 3 will give a detailed description of both modeling tools. Also included in Chapter 3 will be a simple model analyzed in both ERITower and SAP to show that the same results can be provided from both tools. ERITower will be used to analyze the towers with respect to the TIA222-F code under wind and ice loadings only. Results from ERITower regarding wind and ice analysis will be provided in Chapter 4. SAP2000 will be the analysis tool used to model the Taum Sauk tower under seismic loadings. Response spectra and time history loads will be applied using non-linear modal analysis. Details of how the tower is loaded and the results from the analysis under seismic loads will be shown in Chapter 5. Chapter 6 contains results from the parametric analysis of the Taum Sauk tower completed using both ERITower and SAP2000. Chapter 7 contains the summary, conclusions and recommendations for this research.

2 Literature Review

2.1 Introduction

Telecommunication towers, such as the ones used for emergency response systems, require elevated antennas to effectively transmit and receive radio communications. In the absence of tall buildings that antennas can be mounted to, self-supporting (Figure 2.1) and guyed (Figure 2.2) towers tend to be the most economical choice for mounting antennas. These types of towers are generally lightweight in comparison to building a solid structure and are also easier to fabricate and erect. The type of tower used for an application is usually dependent on the design height. “Broadcasting towers generally range from 400 ft to 2,000 ft in height, with those over 600 ft typically being guyed. Towers less than 600 ft will be either self-supporting or guyed, depending on the owner’s preference, budget, and location” (Madugula 2002). Due to space constraints, towers in heavily developed areas tend to be self-supporting while towers in rural areas are often guyed.

While much is known about how the tower will react due to wind and ice loading, very little information exists that models the reaction of these types of telecommunication towers due to seismic loading. “As a result, earthquake-resistant design of these structures cannot simply be extrapolated from simple rules available for buildings” (Amiri 2002). Detailed analysis must be preformed on a model of the tower in question to analyze whether seismic effects are important and whether a more in-depth analysis is required. “In the 1994 edition of CAN/CSA-S37 Antennas, Towers and Antenna-Supporting Structures, a new appendix was introduced to address the seismic analysis of self-supporting telecommunication towers” (Khedr and McClure 1997). The forthcoming ANSI/TIA.EIA Standard 222-G – “Structural Standards for Steel Antenna Towers and Supporting Structures”- also contains detailed revisions in specifying environmental loads and design criteria with a notable increase in emphasis on seismic loads.

With the New Madrid fault line running through the state of Missouri, a detailed examination of how telecommunication towers will react to earthquakes is imperative. Search of the literature does not describe in detail any specific cases of tower damages or failures due to earthquake loading in the U.S (Madugula, 2002). However, this should by no means imply that failures or damages due to seismic activity have not or do not occur. Table 2.1 provides a partial list of several notable tower failures and corresponding failure mechanisms related to more general dynamic loading effects (predominantly wind related).

Figure 2.1. Example of a typical self-supporting (free standing) tower.

Figure 2.2. Example of a typical guyed mast tower.

Table 2.1. Historical record of guyed mast failures due to dynamic effects (Laiho, 1999; adapted from Madugula, 2002).

Date	Location	Tower Type	Failure Mechanism
03/30/1912	Nauen, Germany	200-m mast	Oscillations
03/19/1965	England	384-m cylindrical guyed mast	Oscillations
11/16/1966	Waltham, UK	290-m mast	Cyclic loads (bolts failed)
07/1968	Sioux Falls, SD	60-m	Farmer cut guy wire w/tractor
12/1968	Chacaluco, Argentina	25-m mast	Dish fell and cut guy wire
11/23/1970	Finland	212-m mast	Oscillations (wind and ice)
12/1974	West Germany	-	Oscillations
11/09/1976	Finland	56-m mast	Oscillations
12/28/1979	Sweden	320-m mast	Oscillations
12/31/1979	Czechoslovakia	320-m mast	Oscillations (anchor fatigue)
10/16/1983	Belgium	315-m mast	Oscillations
01/15/1985	Germany	298-m mast	Oscillations, fatigue
12/28/1992	Italy	100-m	Fatigue in legs

2.2 Numerical Simulations

Self-Supporting Towers

In 1994, Mikus conducted one of the first studies using numerical simulations on which many later works would be based, to model the seismic response of self-supporting towers. Six towers were analyzed using three known earthquake accelerograms. Towers ranged in height from 20m to 90m, (65.6ft to 295.3ft). Mikus concluded that by comparing the frequencies of the earthquake records to those of the natural frequencies of the towers, only the four lowest modes were needed in the dynamic analysis using modal superposition. Mikus also found that there was not a pertinent correlation between the results obtained from vertical accelerations alone or by a combined vertical and horizontal acceleration. A comparison of axial leg forces in the tower showed only a 1% difference in the increase of force acting on any leg, thus suggesting that there was no need to include the vertical loading component. Uncertainty in this conclusion could exist however, because more realistic (rather than factored) horizontal accelerograms could cause a greater response in the higher frequency range.

Guyed Masts

In 1994, McClure and Guevara proposed an exploratory numerical simulation of two guyed towers of varying heights subjected to seismic excitation. The first tower was composed of six stay levels with a total height of 350 ft. The second larger tower consisted of seven stay levels and a total height of 1150 ft. Both towers used in that study are in existence and used in industry. Both contain a three-legged latticed steel mast, are pinned at the foundation, and stayed by pretensioned guy wires. The 1940 El Cento and the 1966 Parkfield accelerograms were considered as loading functions to represent dynamic loads containing a wide range of frequencies and several episodes of strong ground motions and single pulse loading with dominant lower frequencies, respectively (Amiri, 1997). The objective of the study was to model the cable geometric nonlinearities and allow for dynamic interaction between the masts and guy cables.

Modeling of the towers was broken up into three basic criteria; the mast, dampening, and guy cables. The modeling of the mast for the shorter tower was made of beam-column elements with equivalent properties. The taller tower used a three dimensional truss to model its mast. Since guy wires possess large geometric nonlinearities that grow as the cables become slack, "sufficiently fine mesh using a large kinematic formulation (but small strains) for the cable stiffness can account for full geometric nonlinearities" (McClure, et. al 1994). The cable dampening and the structural dampening in the masts were not modeled because this would require calculation of too many mode shapes as required to span significant frequencies of the cables and the mast. Results for the smaller tower showed that vertical ground motion could be responsible for causing greater axial force in the mast. In the taller tower the vertical ground motion propagated to the guy wires and amplified the tension in the cables.

Amiri (1997) conducted a study of seismic sensitivity indicators for guyed towers to determine seismic indicators for guyed masts, i.e., to see whether seismic effects will be important in the design of tall guyed towers. Eight existing towers varying in height from 150 to 607 m (492 to1991 ft) (see Table 2.2) were subjected to three different seismic excitations to determine if there were any similarities present in the dynamic tower response.

Table 2.2. Summary of tower characteristics used in Amiri (1997) numerical analysis.

Height (m)	No. of Stay Levels	Location
607.1	9	USA, California, Sacramento
342.2	7	Canada
313.9	5	Canada
213.4	7	Canada
200	8	Argentina, Buenos Aires
198.1	6	Canada, Prince Edward Island
152.4	8	Canada, Alberta, Elk River
150	7	Canada, Alberta, Little Buffalo

Several important seismic sensitivity indicators were proposed, including base shear. For towers shorter than 200m (656ft) results suggested that the base shear is between 40-80% of the total tower weight. Towers over 300m (984ft) produced a total base shear of 15-30% of the total tower weight. The magnitude of the base shear was predicted using the equation:

$$BS = 28300H^{-1.17} \text{ (\% of W)}$$

Where, BS is the maximum percentage of base shear, W is the total tower weight, and H is the tower height in meters. The corresponding graph shown in Figure 2.3 was produced to predict total base shear in towers of varying heights (Amiri, 2002).

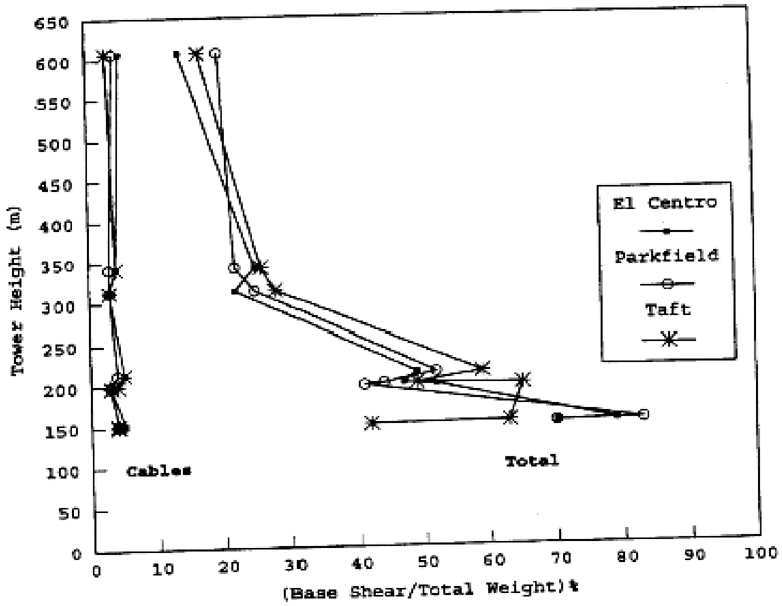


Figure 2.3. Base shear versus tower height (Amiri, 2002).

From the data collected in response to base shear, it was found that towers in the range of 150-300m (492-984ft) may be sensitive to seismic effects. Towers taller than 350m (1148ft) will have naturally low frequencies that match that of typical seismic ground accelerations, which can cause dynamic amplification or resonance to occur. The study also showed that there is only a small contribution to the dynamic component of axial force in the mast members. Axial force varied from 4-10% of the total weight for all of the towers studied except the 200m (656ft) tower. Greater response was shown when the frequencies of the tower match those of the dominant frequencies from the accelerogram. One of the final sensitivity indicators to be identified in that study was the seismic amplification factor of cable tension. This study used two different loading cases to evaluate the effects on the dynamic components of cable tension: horizontal earthquake and combined horizontal and vertical earthquake accelerograms. The seismic amplification factors varied from 30% in the upper clusters of the 607m (1991ft) tower to around 300% in the upper clusters of the 200m (656ft) tower. The typical values for the towers between 150m – 300m (492-984ft) tower heights range from 50% in the upper clusters to 200% in the lower clusters.

Analysis of the cable amplification factors produced some unintuitive results. The 607m (1991ft) tower produced a smaller amplification factor as compared to the smaller towers. Also the 200m (656ft) tower produced the highest amplification factors possible due to it having a slacked guyed system. Another key point of interest was that the combined horizontal and vertical loading cases did not produce higher amplification values than the horizontal accelerogram. Also, the lower clusters in the guyed system were subjected to larger amplification factors than the upper clusters.

2.3 Approximate Static Methods

Self-Supporting Towers

In response to a previous method proposed by Galvez and McClure in 1995, an improved method of their proposed methodology was presented in 1997 with the help of Zaugg(1997). An equivalent static load method was used to model the response of the self-supporting towers used in this study. “The proposed simplified method was based upon the dynamic response of uniform cantilevers, subjected to harmonic base motion” (Galvez et al., 1997). The acceleration profiles for the towers were based upon modal superposition of the effects from three lowest flexural modes of vibration for each tower. Three, three-legged latticed towers were used with heights of 90m, 103m, and 121m (295, 338, and 397ft). Two approaches were used to compare results against one another. The first method used a detailed linear dynamic analysis for each tower subjecting them to 45 differing frequency horizontal accelerograms differing frequency and acceleration-to-velocity ratio (a/v). Results were plotted much like the work of Amiri (1997), but differed in that a general approach was used that could theoretically encompass all of the different accelerograms used for the low, medium and high (a/v) ratios. A proposed static method was also introduced that involved the following six steps:

1. Determining the frequencies and mode shapes for the tower at its lowest three flexural modes
2. Determining the towers acceleration profile
3. Distribution of the mass at the tributary joints on the leg members
4. Calculating the equivalent lateral forces
5. Adding the lateral forces and torsional moments due to the antenna masts
6. Static analysis of the model using the lateral forces and the addition of the lateral forces and torsional moments

A comparison was done between the responses obtained from the proposed static method and the upper bound envelope for the detailed static analysis to each tower. The 90m (295ft) tower produced very conservative results with the average error in the leg members in the range of 30-50% when it was subjected to low and medium a/v ratios. The differences were only 10-30% in the horizontal and vertical bracing. The main point of this study was to demonstrate the feasibility of a static method to compute the axial force responses.

As with all research in relatively new fields, constant advances are made that can improve the accuracy of the original findings. Compounding on the previous study by Galvez et al. (1997), Khedr and McClure (1997) proposed a new approximate static method that would potentially minimize the previous errors for the forces in the tower leg members. Errors in the range of 20-30% were found when comparing the results of the static analysis to a detailed dynamic analysis. The main difference in the approach used was the estimation of the tower acceleration profiles. The previous method used bi-linear acceleration profiles while the current study is based on the response spectrum technique and modal superposition, which are both commonly used in structural dynamics.

The first tower characteristic investigated when subjected to seismic loading was the response to horizontal excitation. To simplify the analysis, the tower was modeled as a linear elastic three-dimensional structure. Beam elements were used for the main legs, while truss elements were used for the horizontal and diagonal members. "Three different earthquake accelerograms were used acting horizontally along one principal direction and classified according to the ratio of maximum ground acceleration and maximum ground velocity (a/v)" (Tso et al. 1992). The earthquakes used were the 1971 San Fernando, which has a low (a/v) ratio, the 1952 California having an intermediate (a/v) ratio, and the 1966 Parkfield having a high (a/v) ratio. The towers were analyzed with the SAP90 software, so the results could be compared to the proposed static analysis results. The results obtained were a significant improvement to those obtained from the previous study of Galvez, et al. (1997). The maximum error produced in the proposed static method did not exceed 25% in the extreme cases and had an average error of 7%.

The vertical response of the towers was first analyzed using the lowest axial mode of vibration. "While most buildings respond to horizontal earthquakes essentially in the lowest lateral mode of vibration, it is not the case for self-supporting towers whose lowest three flexural modes are usually significantly excited" (Mikus, 1994). The towers were analyzed again using the proposed static method from that study. No proposed amplification factors were considered in evaluating the tower member forces, because the tower being evaluated was essentially a linear structure. It was found that the response of the tower to different earthquake accelerograms can be equal to the unit spectral acceleration multiplied by the corresponding spectral acceleration of the earthquake. After using this same procedure on different self-supporting towers it was found that the proposed acceleration profile produced a maximum error of 10% and an average error of only 2%. Although Khedr and McClure (1997) were able to greatly minimize the error obtained from the acceleration profiles that were used, they still suggest performing a detailed seismic analysis when most of the leg members and diagonal members are controlled by seismic loading.

Guyed Masts

As recently as 2002, no approximate static method has yet been proposed for seismic analysis of guyed masts (Madugula, 2002). The primary limitation is that no acceleration profile has yet been created that can account for the mast's lateral stiffness, the interaction between the horizontal and vertical effects, and the towers nonlinear response. Therefore a detailed seismic analysis is warranted when dealing with guyed towers.

2.4 Design Codes for Earthquake Resistant Design

Due to the differences in seismic activity and tower structural standards, most major countries have adopted their own earthquake resistant design code. The majority of the case studies investigated in this literature review were conceived in Canada. Canada relies on the National Building Code of Canada (NBBC), while the United States has produced the ANSI/TIA/EIA-222-F (and forthcoming revision 222-G) for structural standards for antenna supporting structures and antennas.

Section 2.7 of the ANSI/TIA/EIA-222-G (DRAFT), describes in detail how earthquake loads shall be evaluated. The first step in this procedure is to determine the importance factor for the tower in question. Towers in the MoDOT network may be considered to qualify as category three (3) towers used for essential communication such as civil, emergency, and rescue and disaster operations. The corresponding earthquake importance factor is 1.5. Next an appropriate seismic analysis procedure is obtained for the specific tower type in Table 2.3.

Table 2.3. Seismic Analysis Procedure (ANSI/TIA/EIA-222-G (DRAFT) p.42)

Analysis Procedure Method Description [1]	Height Limitations on Analysis Procedure Methods					
	No mass or stiffness irregularities per Table 2-9			With mass or stiffness irregularities per Table 2-9		
	Self-Supporting		Guyed Masts [2]	Self-Supporting		Guyed Masts [2]
	Tubular	Latticed		Tubular	Latticed	
Equivalent Lateral Force, Method 1 in accordance with 2.7.7	50 ft	100 ft	No Limit	N/A	N/A	1500 ft
Equivalent Modal Analysis, Method 2 in accordance with 2.7.8	No Limit	No Limit	N/A	200 ft	600 ft	N/A
Modal Analysis, Method 3 in accordance with 2.7.9	No Limit	No Limit	N/A	No Limit	No Limit	N/A
Time History Analysis, Method 4 in accordance with 2.7.10	No Limit	No Limit	No Limit	No Limit	No Limit	No Limit

Notes:

1. Vertical seismic forces may be ignored for Methods 1, 2 & 3.
2. Method 4 shall be used when the horizontal distance from the base of the structure to any guy anchor point exceeds 1000 feet.

2.5 Procedures for Developing Seismic Response Spectra

Ground motion spectra for dynamic structural analyses may be developed following procedures outlined in the Uniform Building Code (UBC) (1997). As illustrated in Figure 2.4, the simplified UBC spectrum (acceleration, g, versus period, seconds) is defined by a linearly increasing portion up to control period T_0, followed by flat response up to control period T_s, followed by a decaying response to larger periods. Two parameters, or seismic coefficients (C_a and C_v), are required to quantify the spectrum:

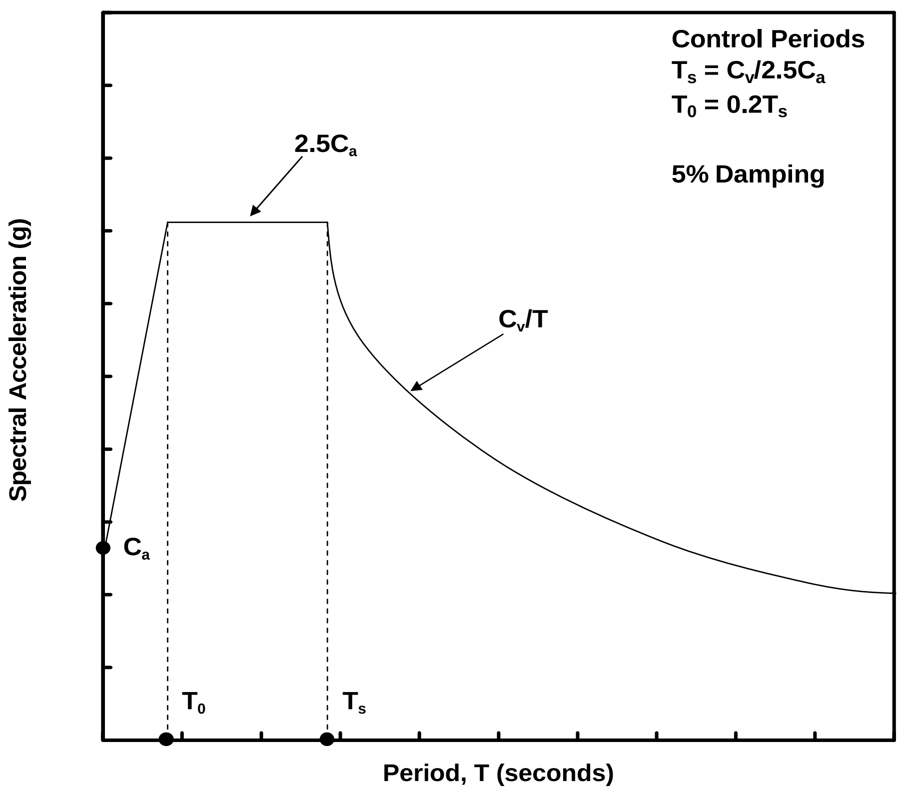


Figure 2.4. Uniform Building Code (UBC, 1997) response spectrum.

Selection of the seismic coefficients is based on the UBC seismic zone for the site under consideration and the near-surface geotechnical (soil/rock) properties. Figure 2.5 is a map showing contours of these seismic zones for the United States. Missouri falls within zones 1, 2A, and 3. UBC soil profile types are summarized in Table 2.4. Soil types

S_A through S_D are defined based on measured or estimated shear wave velocity (V_s), standard penetration test blow count (N), or undrained shear strength values. Soil type S_E is defined by these values as well as the existence of any clay layer thicker than 10 ft with plasticity index PI > 20, water content w > 40%, and undrained shear strength s_u < 500 psf. Soil type S_F defines a deposit vulnerable to potential liquefaction or collapse and requires special site specific treatment. Soil properties selected for assigning UBC soil type are those that are considered most representative of the site from the ground surface to a depth of 100 ft (30 m).

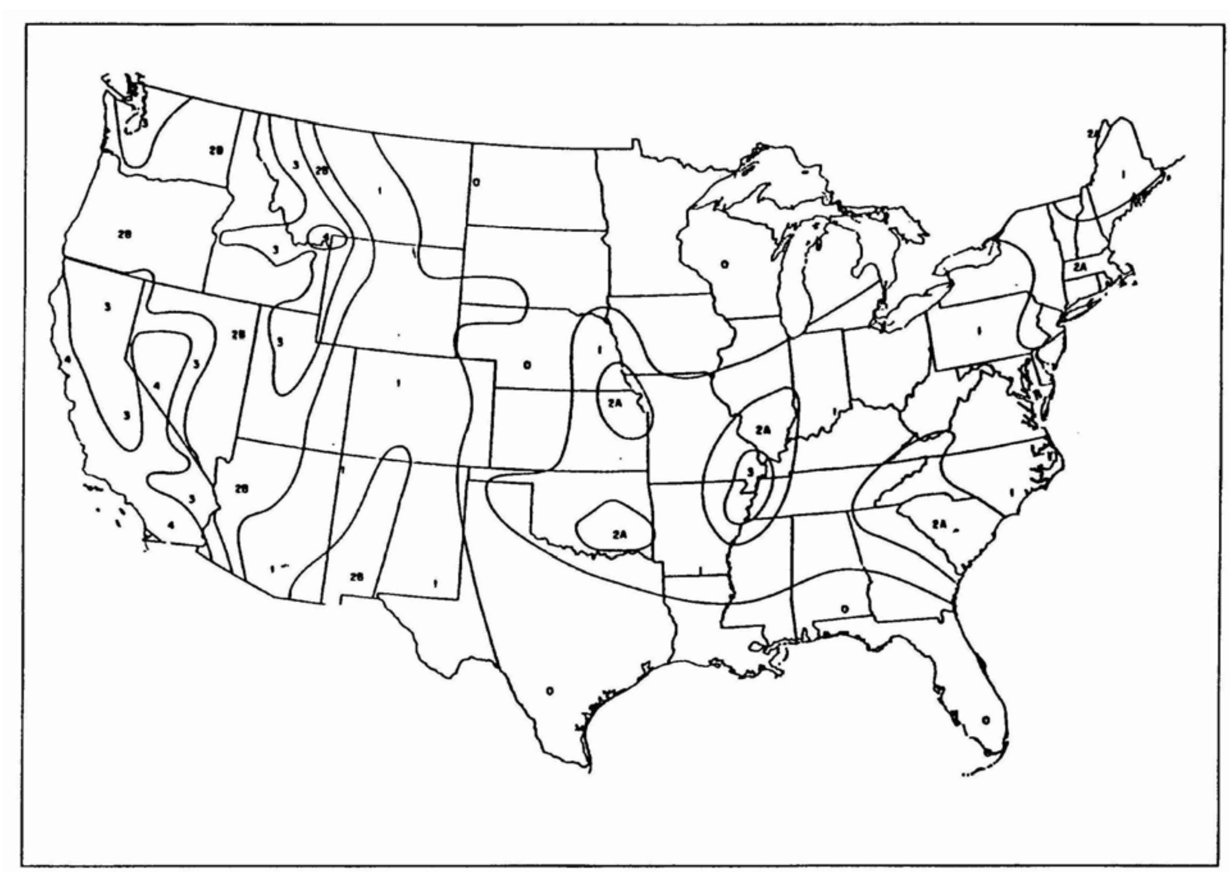


Figure 2.5. UBC seismic zones for the United States. Note: The contour intervals originating in Southeast Missouri are 3 and 2A. The majority of Missouri falls within contour 1. The small contour in Northwest Missouri is 2A.

Table 2.4. UBC soil profile types.

Soil type	Description	Shear Wave Velocity, V_s ft/s (m/s)	SPT $(N_1)_{60}$	Undrained Strength, s_u psf (kPa)
S_A	hard rock	> 5000 (> 1500)	--	--
S_B	rock	2500–5000 (760-1500)	--	--
S_C	soft rock dense granular soil; very stiff cohesive soil	1200-2500 (360-760)	>50	>2000 (>100)
S_D	dense granular soil; stiff cohesive soil	600-1200 (180-360)	15-50	1000-2000 (50-100)
S_E	loose to med granular soil; soft to med cohesive soil	< 600 (< 180)	< 15	< 1000 (< 50)
S_F	special case; vulnerable to collapse or liquefaction (site specific analysis necessary)			

Given the seismic zone and soil profile type, the seismic coefficient C_a may be obtained from Table 2.5 and C_v may be determined from Table 2.6. Once the seismic coefficients are determined, the parameters T_s and T_0 defining the remainder of response spectrum can be calculated as

$$T_s = \frac{C_v}{2.5C_a} \qquad T_0 = 0.2T_s \tag{2.1}$$

Response spectra generated in this manner may be used directly as input for dynamic structural analysis using SAP 2000. Equivalent time-domain loading functions (accelerograms) may be simulated from the UBC response spectra using the procedures described in Tulasi (2005).

Table 2.5. UBC seismic coefficient C_a

Soil type	Zone 1	Zone 2A	Zone 2B	Zone 3	Zone 4*
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.44N_a$
S_F			site specific evaluation required		

* N_a = near source factor

Table 2.6. UBC seismic coefficient C_v

Soil type	Zone 1	Zone 2A	Zone 2B	Zone 3	Zone 4*
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F			site specific evaluation required		

* N_v = near source factor

UBC response spectra were developed for dynamic structural analysis of the guyed communications tower at Taum Sauk (soil profile type “rock;” seismic zone 2A). Corresponding parameters are

$$C_a = 0.12\,,\ C_v = 0.12\,,\ T_s = C_v / 2.5C_a = 0.4\text{s}\,, T_0 = 0.2T_s = 0.08\text{s}$$

and the response spectrum is shown as Figure 2.6a. Figure 2.6b shows an equivalent time-domain accelerogram for a “short” duration (10 s) seismic event. Figure 2.7a and 2.7b show similar traces for a “long” duration event. These spectra and accelerograms were used as input loading functions to SAP2000 modeling software as described in the following chapters.

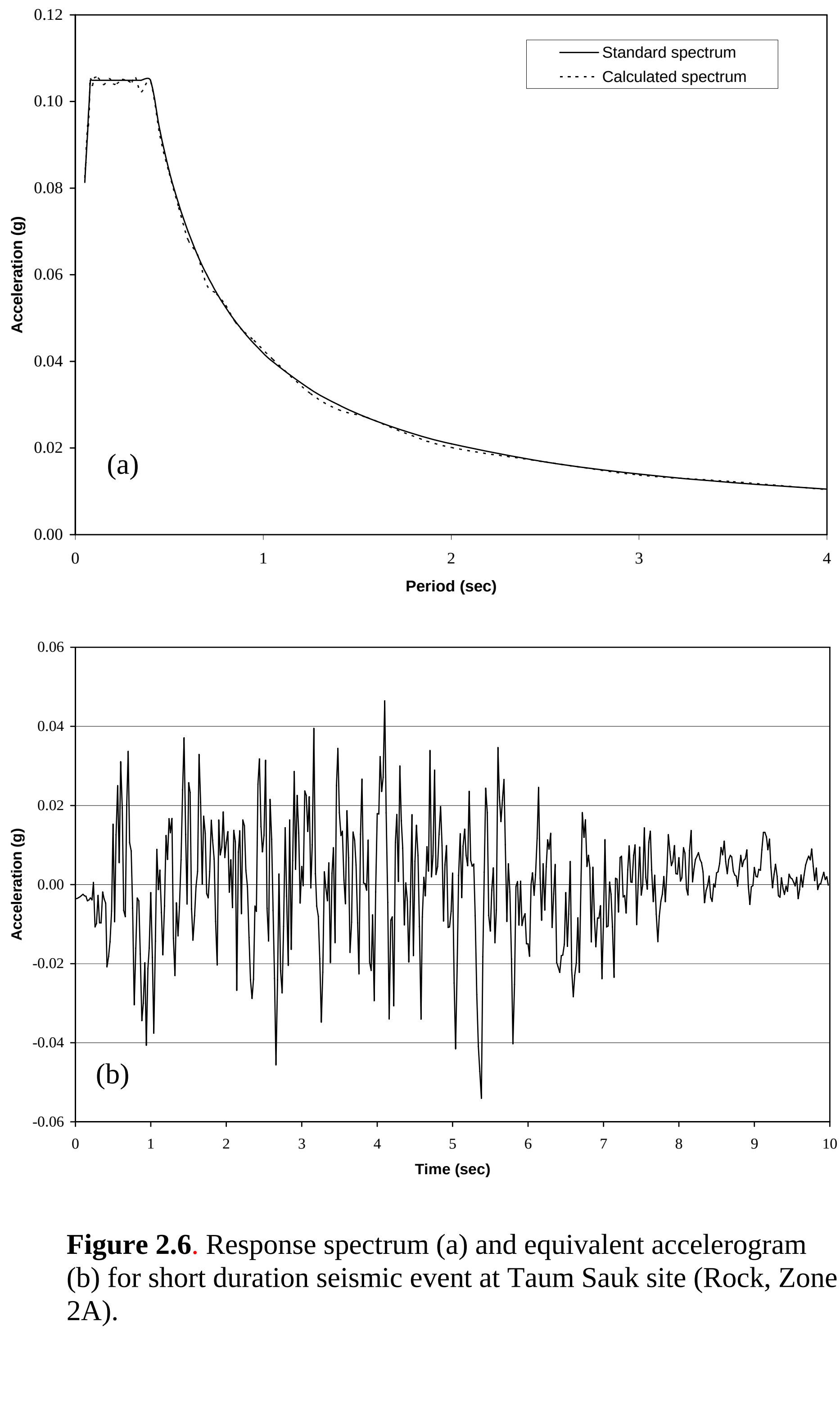
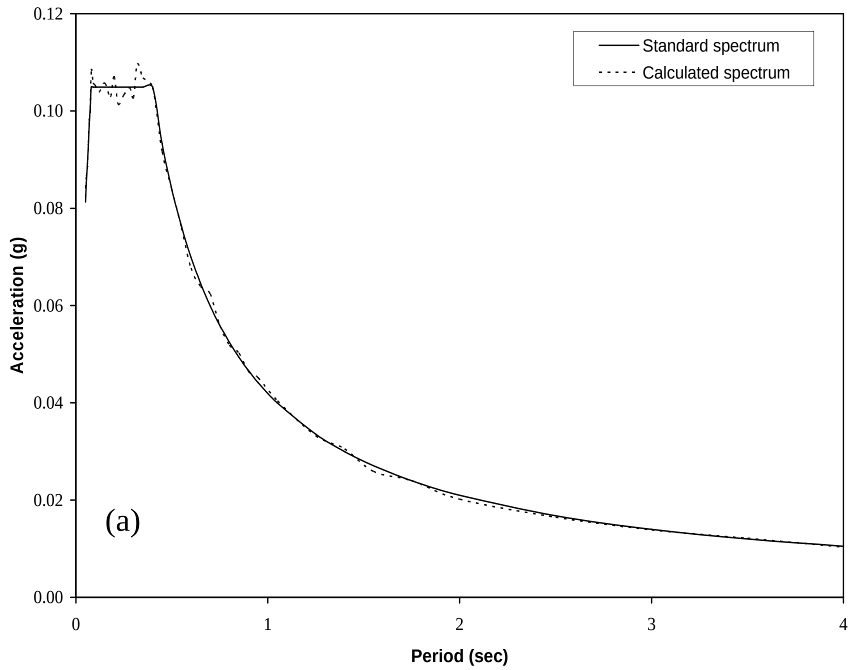


Figure 2.6. Response spectrum (a) and equivalent accelerogram (b) for short duration seismic event at Taum Sauk site (Rock, Zone 2A).



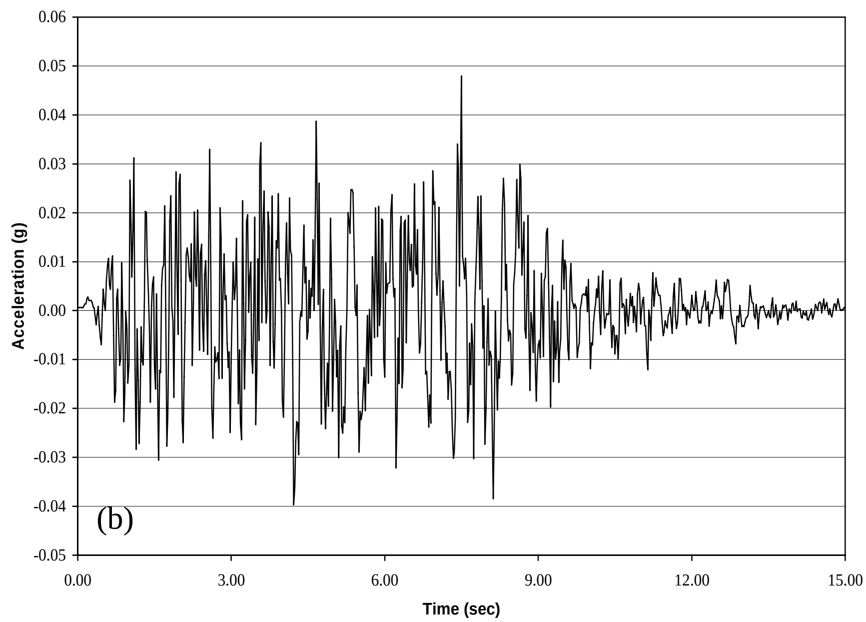


Figure 2.7. Response spectrum (a) and equivalent accelerogram (b) for long duration seismic event at Taum Sauk site (Rock, Zone 2A).

3 Modeling and Analysis Tools

3.1 Introduction

Radio towers, such as the Kansas City and Taum Sauk towers, are complex structures with many members. The large number of members makes these towers difficult to analyze by hand, due to the many calculations necessary. To make analysis run more quickly and accurately, computer software has been designed to do finite element and modal analysis. These tools allow for a model to be created fairly quickly and for member sizes and connection types to be changed easily. They also allow for multiple loads and load combinations to be applied to the structure at the same time. For Analysis of the Kansas City and Taum Sauk towers, two modeling and analysis tools have been chosen. ERITower was chosen to compute wind and ice loadings on the tower. It follows the TIA222 code and has all past versions of the code programmed into the software. ERITower is not capable of dynamic analysis, thus making it necessary to use another modeling tool. SAP2000 will be used to model the Taum Sauk tower and analyze it using non-linear modal analysis.

The rest of this chapter will discuss each modeling tool in more depth. Also, an analysis of a simple model will be done using both softwares and the results will be compared. This comparison should show that the two softwares produce accurate results.

3.2 ERITower

ERITower is a structural analysis program used to analyze 3 and 4 sided towers for ice and wind loads. Towers can be either guyed or self-supporting. The program is a compilation of spreadsheets that aid in the modeling of geometry of the tower, and application of external loads such as antennas, dishes and feedlines. ERITower analyzes the towers using the TIA-222-F standard or any of the previous versions of the TIA/EIA standards. For steel analysis, the program uses the AISC ASD 9th edition. Linear and nonlinear (p-delta) analyses can be performed to determine the displacements and forces in the structure. Once analysis has been performed, ERITower creates an extensive report consisting of all inputs into the software and results for the tower. The results include stresses in each member of the tower and whether or not the members fail or pass with respect to the standards and codes that were applied.

The user interface of ERITower is very user friendly. The input menu opens to a page for input of a code. Along the top of the menu are tabs for all inputs necessary to create a tower model including Geometry, Guy Data, Discrete Loads, Dishes, etc. The Code tab allows for input of specific code and other wind and ice requirements. All versions preceding the TIA222-F code and the TIA222-G, which is under review, are included in this software. The code to be used can be selected from a pull down menu and then is automatically applied to the tower. There is also an input for ice requirements and wind requirements that can be found depending on the state and county where the tower is located. There is a seismic input; however the seismic analysis part of

ERITower is not functional for this version of the software. Figure 3.1 shows a screen shot of the Code and how the user interface is set up.

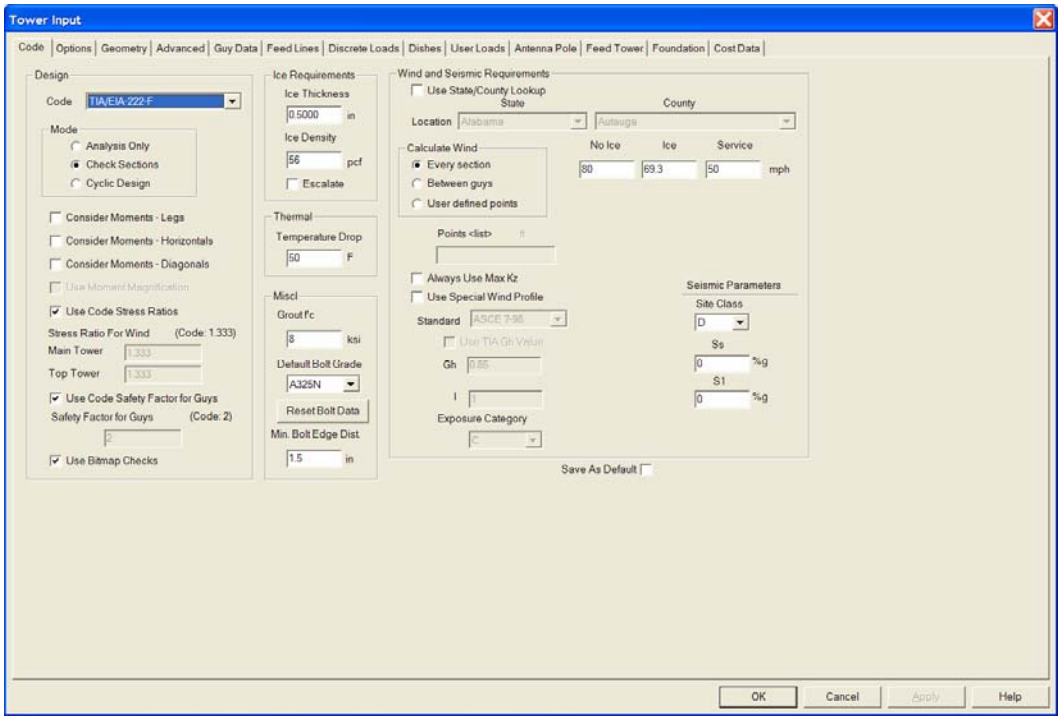


Figure 3.1. Screenshot of Code input interface on ERITower

The “Geometry” tab takes the user to the interface that allows the tower model to be created (See Figure 3.2). Here tower type can be chosen depending on if the tower is free standing or guyed, and 3 or 4 sided. The tower height and number of sections can then be defined. Using a spreadsheet format, ERITower allows the user to define each member size and bracing type. Pre-defined sections are already loaded into ERITower’s database, however there is an option to create original sections as needed.

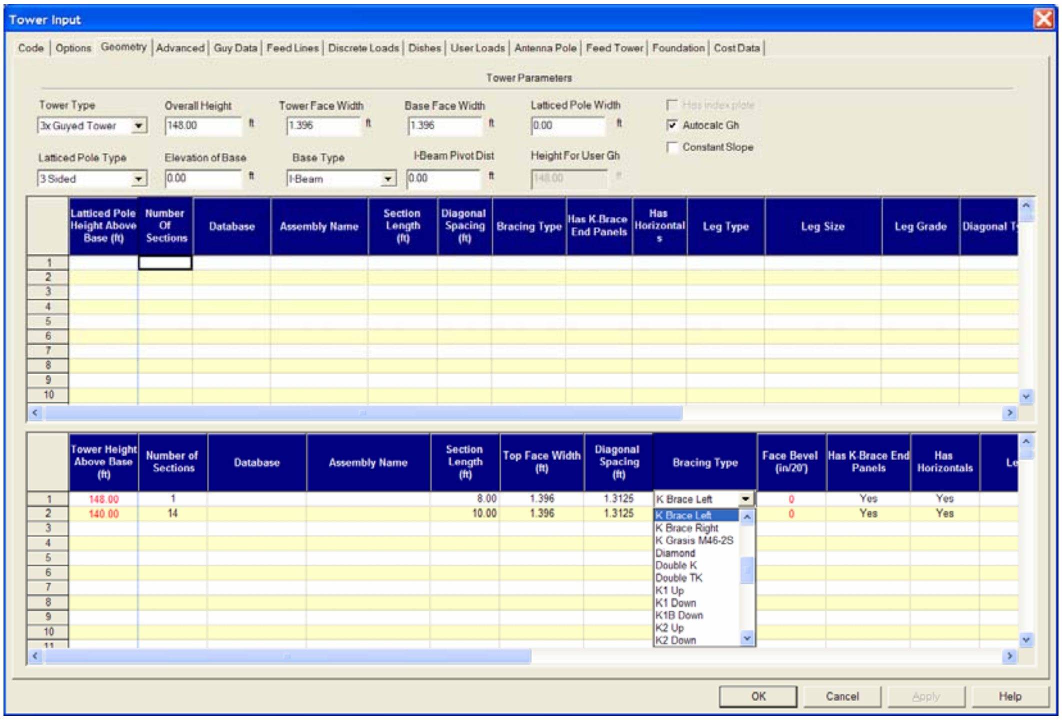


Figure 3.2. Screenshot of Geometry input interface on ERITower

Another helpful tab on the Tower Input menu is the "Discrete Loads" tab (See Figure 3.3). Satellite dishes and antennas are two examples of external loads applied to the tower. ERITower has a database of typical manufacturers and models of dishes and antennas. These are helpful when the manufacturer and model of an attachment are known. The model can be chosen from a drop down menu and applied to the tower very simply. If the attachment model is unknown, then it can also be manually modeled by defining its physical properties.

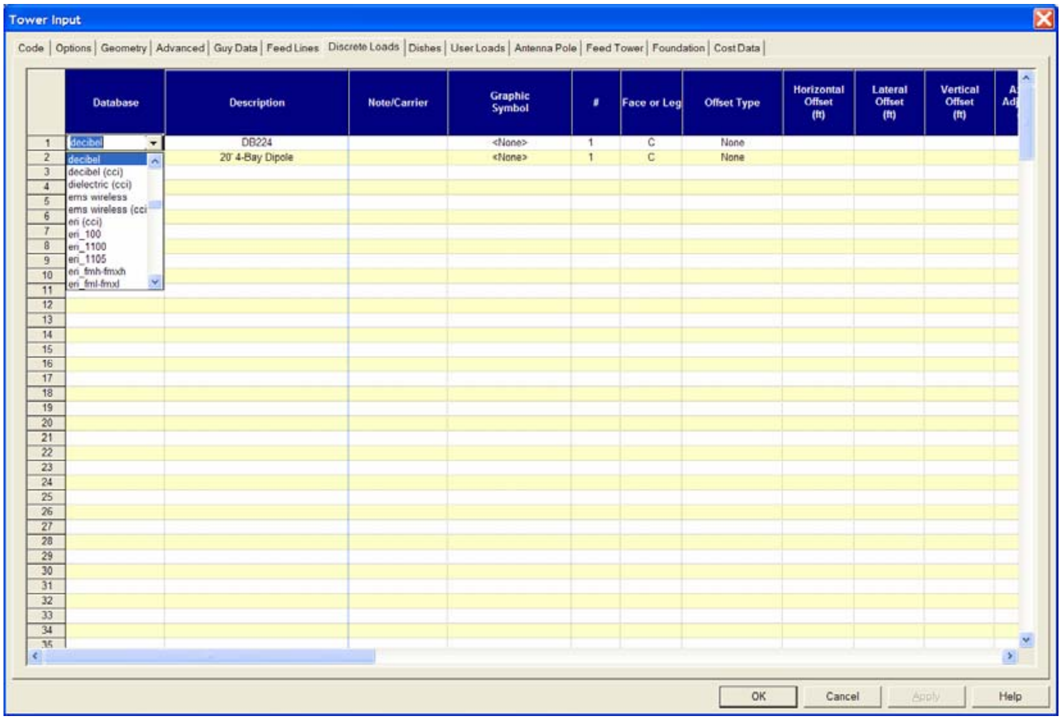


Figure 3.3. Screenshot of Geometry input interface on ERITower

Other inputs included on the Tower Input menu include Tower Options, Advanced Options, Guy Data, Feed Lines, Discrete Loads, User Loads, Antenna Pole, Feed Tower, Foundation, and Cost Data. All of these interfaces are spreadsheets similar to those outlined above, that can be modified to create the tower model.

Once the tower is created, it can be analyzed by ERITower with respect to the defined code. ERITower will run an analysis on the tower and create an output report. The report consists of the inputs for the tower, member stresses due to loading, and a detailed list of whether each member passed or failed with respect to the code. A member is said to pass if it is loaded at less than 100 % of its allowable capacity. Also included in the report is a print out of the model outlining details of the tower.

3.3 SAP2000 v9.0

SAP is a static and dynamic structural analysis program that includes linear and non-linear analysis capabilities. Seismic analysis can be performed using SAP, and the ground motion can be modeled using spectrum or time history functions. Of particular interest for this project were the dynamic modeling capabilities of SAP, which can be performed using response spectrum analysis, time history analysis, and combinations of loading scenarios. Modal analysis was performed using Eigenvector analysis for response spectrum function and Ritz vector for time history function. SAP allows the user to input the response spectrum function. Preprocessing in SAP utilizes a graphical interface for defining the tower geometry and properties of members and for defining loads and load combinations. Post processing provides output for internal forces and moments, displacements, mode shapes, and design checks.

SAP has a fairly complicated user interface, however it allows for more complicated models and analyses of structures. To begin in SAP, the user is required to create a coordinate/grid system, either rectangular or cylindrical coordinates, in which the model of the structure will be drawn. The framework of the model is drawn by defining the grid. Members are then drawn from intersections of the gridlines to create the basic model. Once the simple model had been drawn, member sizes and joint releases can be defined. By picking a member, a section can be assigned by selecting the section from a drag down menu.

Following the defining of members and sections, loads and loading combinations need to be defined. Loads can be defined by using one of the pull down menus at the top of the user interface. Selecting a point or member, loads can be assigned similar to the way sections are defined. Along with simple point and distributed loads, loading functions can be defined also. Important functions that can be loaded on the tower using SAP are time history functions and response spectrum functions. These functions can be imported into SAP and then a modal analysis of the tower can be executed. This is especially important for the research performed on the Taum Sauk tower, as these two types of functions are how earthquake loading can be modeled.

3.4 Simple Example Using SAP and ERITower

SAP and ERITower are both very useful tools when analyzing towers under different loading conditions. ERITower is specifically designed for tower analysis, however it is not able to complete seismic analysis. SAP is a general structural analysis tool that can be used to model many different and elaborate structures. Not only can it analyze wind and ice loads, it can also perform dynamic analysis. This allows for structures to be analyzed under earthquake loads.

Though the tools are different they still produce the same output when identical loading conditions are placed on the tower. To show that SAP and ERITower work correctly, a simple model was created using each software. The models and load applied to the models are identical.

The model is the bottom 30 feet of the Kansas City tower. The tower is fixed at the three connection points to the foundation. Figures 3.4 and 3.5 show the structure modeled in ERITower and SAP, respectively. A 100 pound load is applied normal to one of the faces of the structure, 30 feet above the ground and centered on the face. Figure 3.6 shows the load applied to the tower in SAP. ERITower does not show the loads on the tower. The loads are defined in the report output by the software instead.

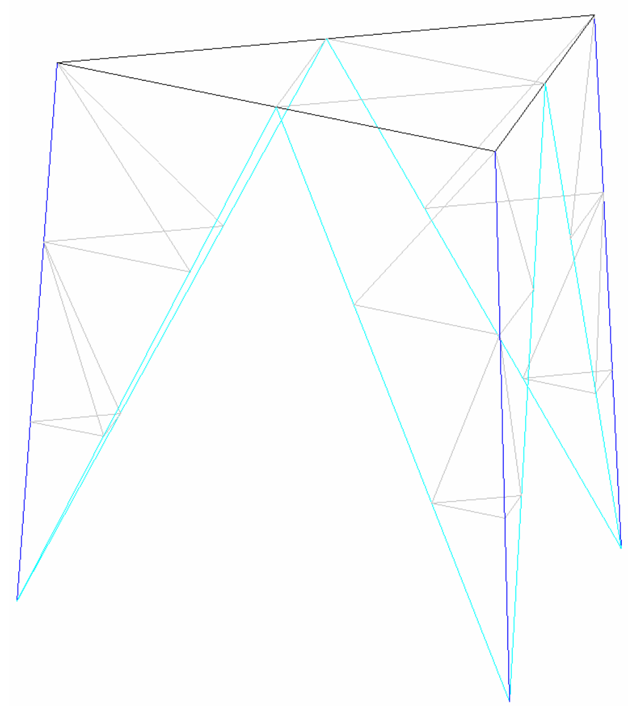


Figure 3.4. ERITower model of simple structure

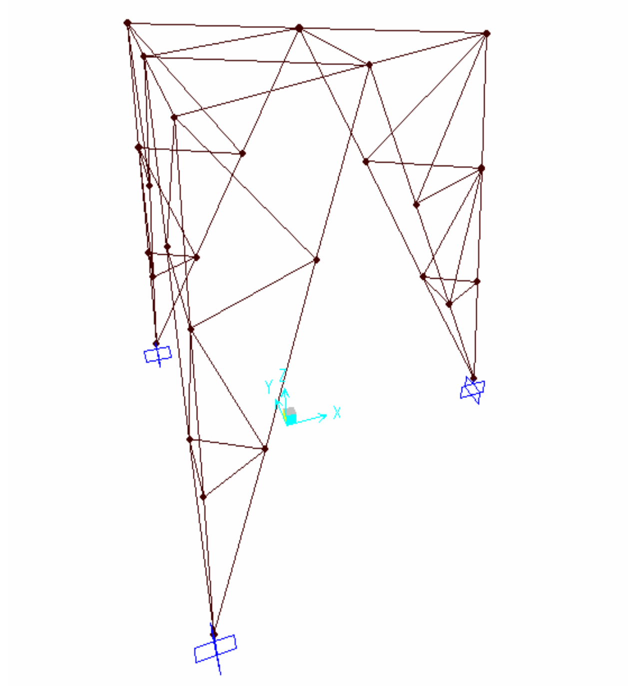


Figure 3.5. SAP model of simple structure

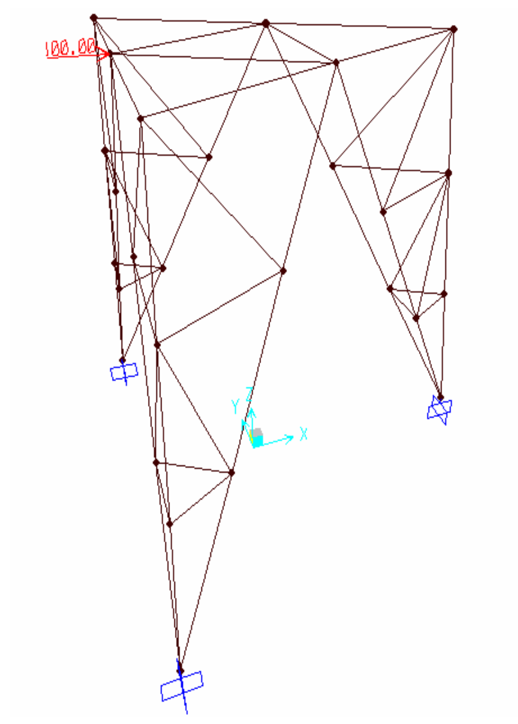


Figure 3.6. Simple model with 100 lb load

3.5 Results

The two simple versions of the tower were loaded using the same 100 pound load placed at 30 feet up. The most simple way to determine if ERITower and SAP produced the same results would be to compare the reactions. For ERITower the reactions are not given specifically for each connection to the ground. However, they are summed and a full reaction in the x direction is calculated. The SAP results give the reactions at each ground connection in the x, y and z directions. For this comparison the reactions in the x direction will be used as they should also add up to equal the magnitude of the load.

For ERITower, the tower was found to have a shear reaction of 100 lb and a moment of around 3000 lb-ft. These reactions seem correct as the only force on the tower was the 100 lb load at 30 feet elevation. This would make the overturning moment of the tower 3000 lb-ft at the base of the tower. SAP only gave reactions in the x, y and z planes. No moments were calculated. In the x direction, the reactions at the three legs were -242.47, -242.47 and 384.94 pounds. When these reactions are summed the total shear force in the x direction is found to be 100 pounds.

Given that the two modeling tools produced the same simple results for this fairly simple model, it will be assumed that ERITower and SAP are capable of the analyses that will be done during this project. From this analysis of the modeling tools, progress will now be made in the development of the Kansas City and Taum Sauk tower models. Once the towers have been modeled, wind, ice and earthquake load analysis will be done using the TIA222-F code.

4 Analysis Under Ice and Wind Loading

4.1 Introduction

Two major environmental factors affecting communication towers in the state of Missouri and other areas of the country are ice and wind loads. The Kansas City tower and the Taum Sauk tower were both analyzed for ice and wind loading conditions using ERITower. ERITower is a structural analysis program used to analyze 3 and 4 sided towers for ice and wind loads. Towers can be either guyed or self-supporting. The program is a compilation of spreadsheets that aid in the modeling of geometry of the tower, and application of external loads such as antennas, dishes and feedlines. ERITower analyzes the towers using the TIA-222-F standard or any of the previous versions of the TIA/EIA standards. For steel analysis, the program uses the AISC ASD 9th edition. Linear and nonlinear (p-delta) analyses can be performed to determine the displacements and forces in the structure. Once analysis has been performed, ERITower creates an extensive report consisting of all inputs into the software and results for the tower. The results include stresses in each member of the tower and whether or not the members fail or pass with respect to the standards and codes that were applied.

4.1.1 Kansas City Tower Model Details

The Kansas City tower is a 250 foot, 3-sided, self-supporting tower. Drawings for this tower, dated August 19, 1965, were obtained from MoDOT and used to model it using ERITower v3.0. The model was created using the member sizes shown on the drawings. Some members were not labeled clearly in the drawings so a trip was made to Kansas City to measure some of the missing member data. While measuring some members it was found that the tower had been built with member sizes that differ from the design drawings. The analysis performed, however, did not take into account this discrepancy as we could not measure every member on the tower. Member sizes that were missing were taken from field measurements and approximations for members that were out of reach were made. The discrepancies in member sizes were only based on diameters and differed from the drawings by at most 0.75 inches. Wall thicknesses of the tubes were not accounted for since calipers were not available to determine good measurements. Approximations were based on the relations of sizes of other members in the same areas.

For this tower, there were 12 separate sections. The sections were labeled T1 to T12 with T12 being 30 feet tall and the remaining sections being only 20 feet. Antennas and feedlines were placed on the tower based on information given by MoDOT. Two DB 224 antennas were placed on the tower on the same leg. The upper DB224 dipole antenna is mounted so that the top is 14" below the top of the tower. Specs show the DB 224 to have an overall mast height of 21 feet. The next-to-top DB224 dipole antenna is mounted with a 14" clearance from the bottom of the antenna above it. Three other antennas (8 foot whips) were not included on the model. Photographs of the tower and the corresponding ERITower model are shown in Figure 4.1a and 4.1b, respectively. Results of the wind and ice analysis are summarized next.

4.1.2 Taum Sauk Tower Model Details

Tower structural drawings were used to model and analyze the Taum Sauk tower for wind and ice using ERITower v3.0. The tower was broken up into 15 sections labeled T1 up to T15. All sections are 10 feet tall. Each section has 1 ¼ inch by 14 gage pipes as legs and 7/16 inch diameter solid rods as K-bracing. The spacing for the K-bracing is 1.3125 feet. The guys were attached to the tower at 30, 60, 100 and 140 feet mounted on the corners. The guys attached at 140 feet were a 3/8-inch cable while the remaining guys were 5/16 inches in diameter. The connection points on the ground were 60 feet from the base of the tower for the lower two guys and 120 feet for the upper two. Finally the initial tension was determined as a percentage of the capacity of the cables from the drawings and input into the model. For modeling purposes, an assumption was made for the antennas on the tower. A DB 224 was placed at a start height of 130 feet, and a 20 foot 4 bay dipole was placed on top of the tower. Both of the antennas were on the same face of the tower. Finally, two feedlines were added on the same face as the antennas running the entire length of the tower. Figure 4.2 shows a model of the tower.

4.2 Loading

The two towers were analyzed using loadings from the TIA-222 standards. The Kansas City tower was loaded first with the TIA-222-C code, which is the code used for the original design of the tower. The tower had discrepancies in member sizes from as built and blueprints, therefore it was necessary to determine if the tower was originally built to standards. It was then loaded with respect to the TIA-222-F code which is the current standard for radio towers. The Taum Sauk tower was only analyzed using the TIA-222-F code. ERI Tower allowed for different inputs determined by the code and where the tower was located. Following are the loading specifications used for each tower.

4.2.1 Loading of Kansas City Tower

Using Standard TIA-222-C: ERITower v3.0 allows the engineer to choose which code to use for analysis. The Kansas City tower was first analyzed using the TIA-222-C standard to determine if it was up to code for the standard for which it was originally designed. The code uses a safety factor of 2.5. Ice thickness and density were assumed to be 0.5 inches and 56 pcf, respectively. Wind loading was calculated for every section of the tower. The original structural drawings indicated the tower was designed for a 30 psf wind load. Accordingly, wind zone A for the TIA-222-C code was chosen since it is the only zone that includes a 30 psf wind load. The wind zones are defined to include pressures as follows: A (30, 35,50 psf), B (40, 48, 65 psf), or C(50, 60, 85 psf). A wind multiplier of 1.0 was used when ice was not included in the load combination and a multiplier of 0.75 was used when ice is included. The load combinations are then found to be:

$D + 1.0(W_O)$: without ice
$D + 0.75(W_I) + I$: with ice

Where D is dead load, Wo is wind load on the structure without ice, WI is wind load on the structure with radial ice, and I is the weight of the ice.

<u>Using Standard TIA-222-F:</u> The K.C. tower was also analyzed using the TIA-222-F standard to determine if it was up to the current code. The TIA-222-F standard uses a safety factor of 2.0. For ice calculations, a thickness of 0.5 inches and a density of 56 pcf were used as defined in the TIA-222-F code in Annex H. Wind speeds were auto calculated using the state/county look-up provided. Clay County, Missouri was used and wind speeds of 75mph (no ice), 64.9519mph (with ice), and 50mph (service) were calculated. The wind profile for this code comes from the ASCE 7-98 and exposure category C was used. The load combinations are then found to be:

$D + 1.0(W_O)$: without ice
$D + .75(W_I) + I$: with ice
D = Dead weight of structure
W_O = design wind load on the structure, without ice
W_I = design wind load on the structure, with ice
These combinations are defined in the TIA-222-F section 2.3.16.

For the stress checks done by the software on the steel members, the AISC ASD 9th edition is used. This code is used since the steel is assumed to be cold rolled. A stress ratio of 1.0 is used in the checks with respect to equations H1-1, H1-2 and H1-3 in the ASD manual.

The ASD equations are as follows:

(H1-1), $$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}} \le 1.0, \qquad (4.1)$$

(H1-2), $$\frac{f_a}{0.60 f_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \le 1.0, \qquad (4.2)$$

When $f_a/F_a \le 0.15$, Equation use (H1-3) instead of (H1-1) and (H1-2):

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \le 1.0, \qquad (4.3)$$

Where:

Fa = axial compressive stress permitted if axial force alone exists

Fb = compressive bending stress permitted if bending moment alone exists

F’e = Euler stress divided by factor of safety

fa, fb = computed axial and compressive stresses, respectively

Cm = Coefficient determined in section H1 of the ASD steel construction manual

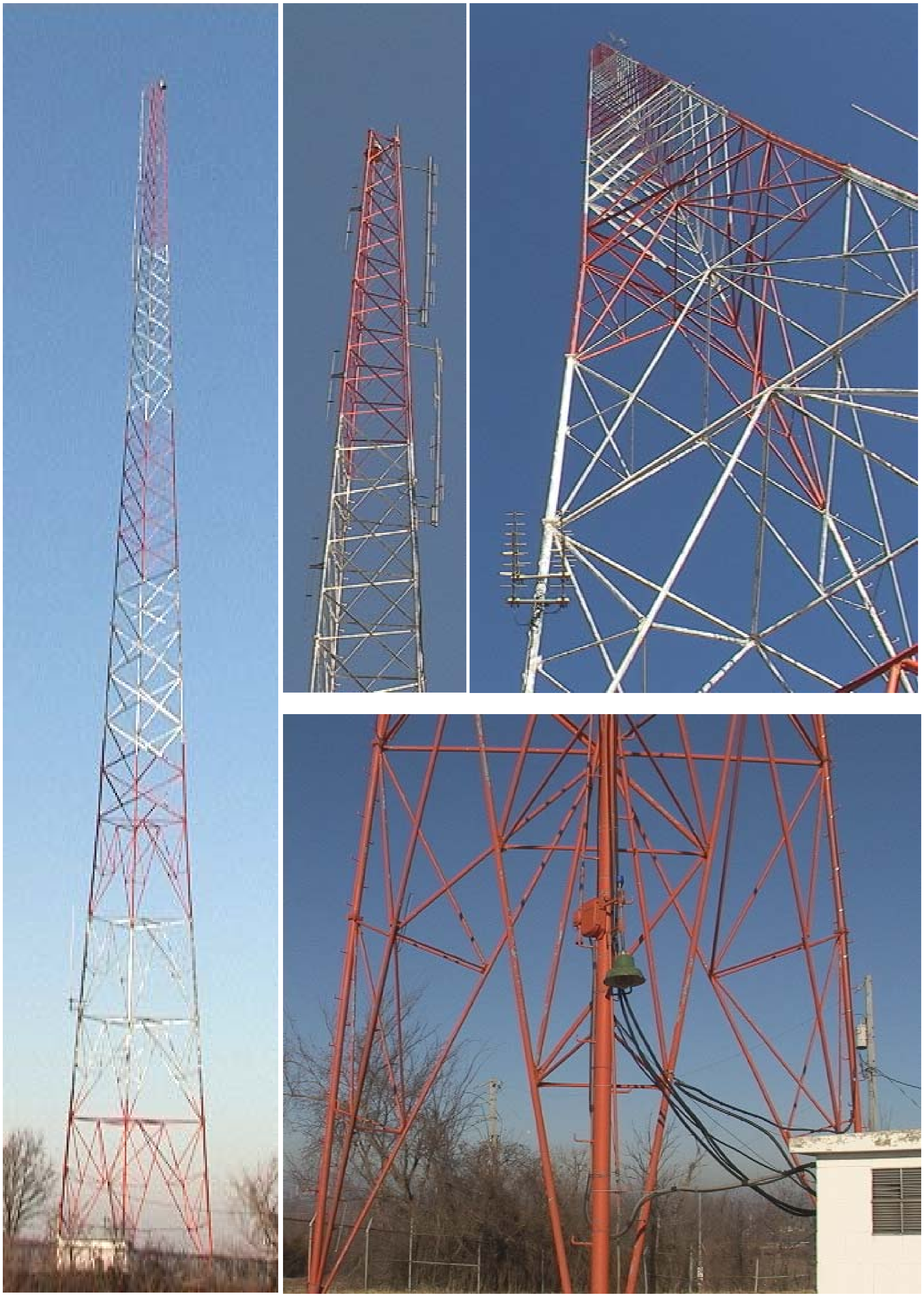
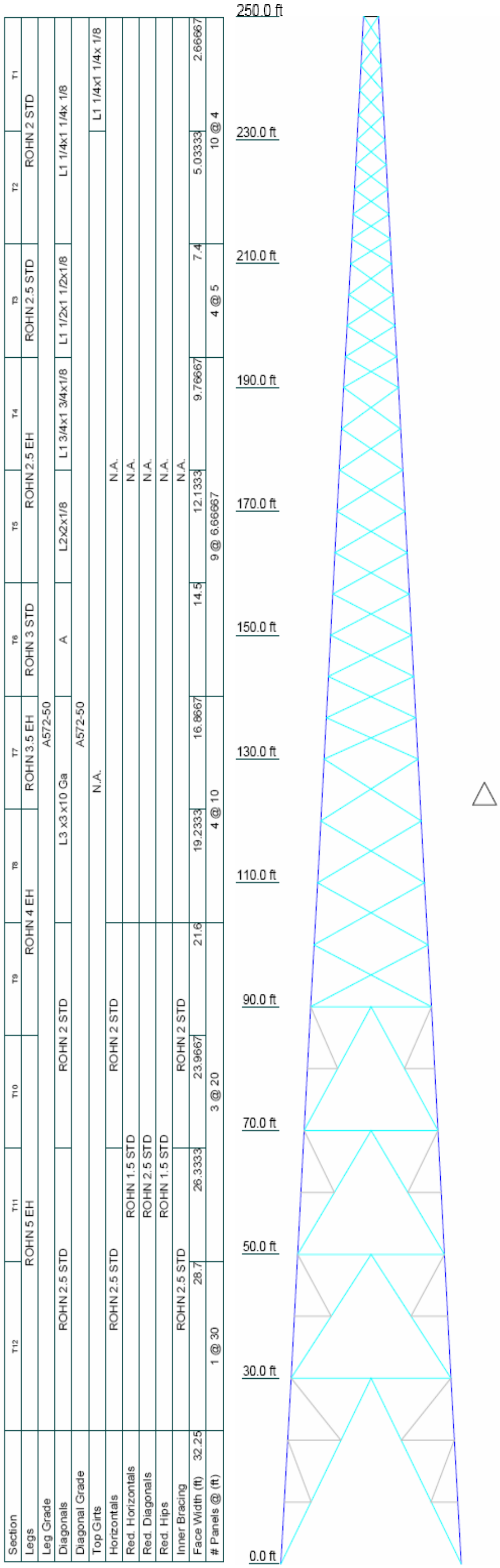


Figure 4.1. (a) Select Photographs of Kansas City Tower



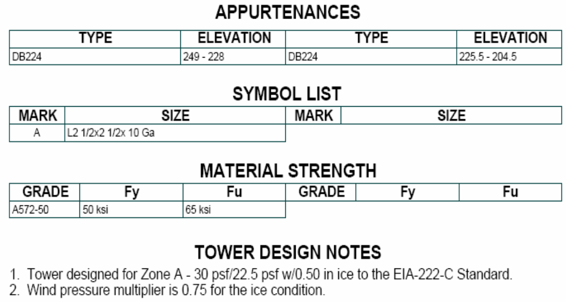


Figure 4.1. (b) ERITower Model of the Kansas City Tower

4.2.2 Loading of Taum Sauk Tower

The tower was analyzed using the TIA-222-F standard to determine if it was up to the current code. The TIA-222-F standard uses a safety factor of 2.0. For ice calculations a thickness of 0.5 inches and a density of 56 pcf were used. Wind speeds were auto calculated using the state/county look-up provided. Wind speeds of 70 mph (no ice), 60.6281mph (with ice), and 50mph (service) were assigned. The wind profile for this code comes from the ASCE 7-98 and exposure category C was used. The load combinations used are:

$D + 1.0(W_O)$: without ice
$D + .75(W_I) + I$: with ice
D = Dead weight of structure
W_O = design wind load on the structure, without ice
W_I = design wind load on the structure, with ice
These combinations are defined in the TIA-222-F section 2.3.16.

For the stress checks done by the software on the steel, the AISC ASD 9th edition is used. This code is used since the steel is assumed to be cold rolled. A stress ratio of 1.0 is used in the checks with respect to equations H1-1 in the ASD manual.

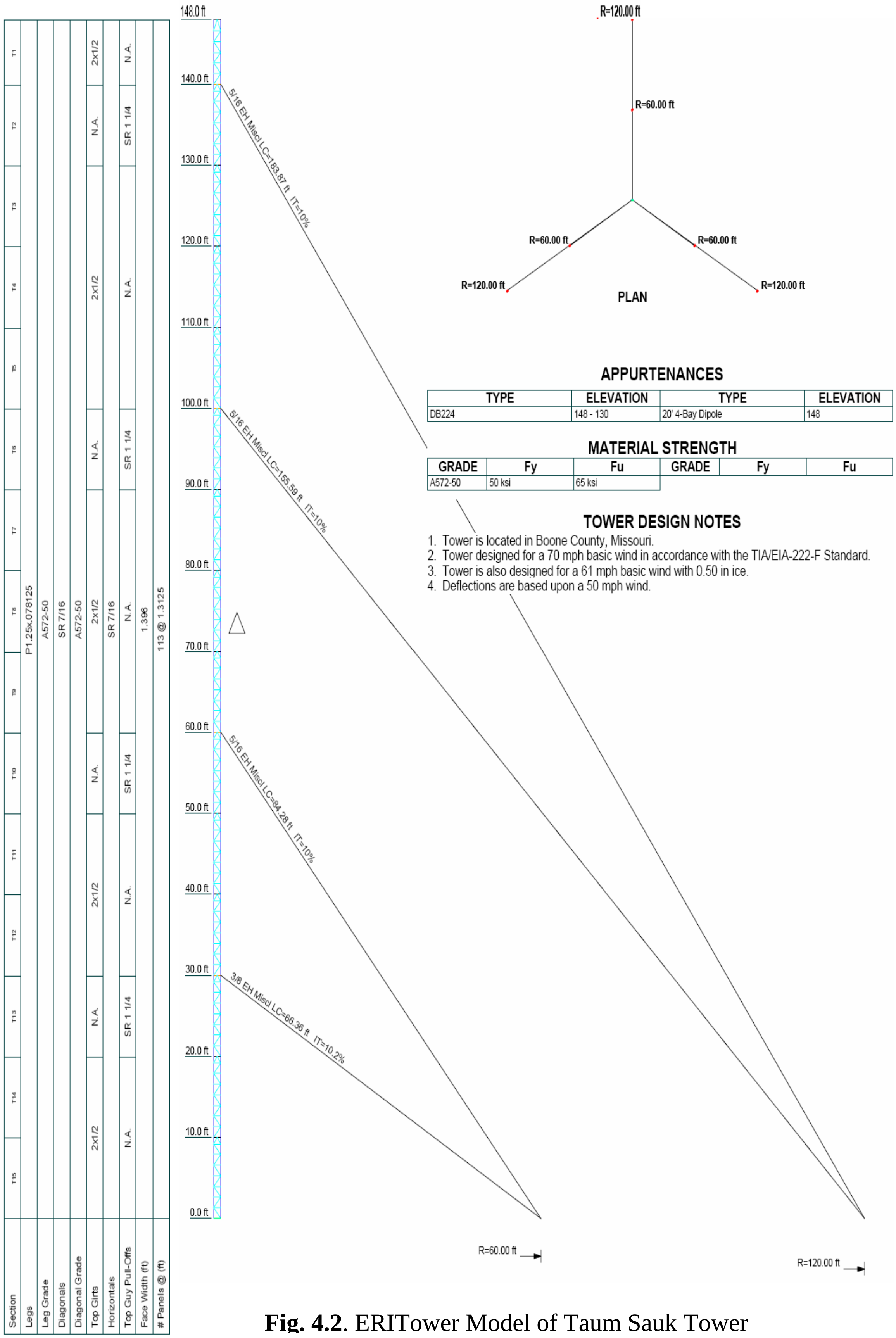


Fig. 4.2. ERITower Model of Taum Sauk Tower

4.3 Kansas City Tower Analysis

After the tower was modeled, it was analyzed using the TIA-222-C and TIA-222-F standards, respectively. For the stress checks in each member, the ratios of the actual versus allowable loads and pressures were used. The equation that follows was used for both the TIA-222-C and TIA-222-F checks.

$$\frac{Combined \quad Stress \quad Ratio}{Allowable \quad Stress \quad Ratio} \leq 1.0 \qquad (4.4)$$

From this check, a critical member can be selected for each type of component. The actual combined stress ratios can be divided by the allowable stress ratios (ASR) to determine the percent capacity of each section. This capacity is what determines if the section, and eventually the tower, passes or fails. Tables 4.1 and 4.2 show the critical components in the tower and their capacities using the TIA-222-C and TIA-222-F standards, respectively.

It can be seen from Table 4.1 that the critical component in the tower is a diagonal located in section T8. The diagonal is at 101.7 % capacity. The tower has an overall rating of 101.7%.

It can be seen from Table 4.2 that the critical component in the tower is a diagonal located in section T10. The diagonal is at 1177% capacity and fails. The tower is now said to have a rating of 1177%, which is much greater than the allowable 100%. Therefore, the tower fails when the stress ratio is equal to 1.0. For towers built for the TIA-222-F code, however, there is an option to increase the allowable stress by 1/3 for the load combinations if the tower is less than 700 feet tall. This increase in the allowable stress is stated in the TIA-222-F, section 3.1.1.1. This makes the allowable stress ratio

(ASR) become 1.333 instead of 1.0. When the Kansas City tower is analyzed for the F standard with a stress ratio of 1.333, the tower receives a rating of 88.9% which passes. The critical components for the tower with a stress ratio of 1.333 can be seen in Table 4.3.

Table 4.1. Summary of TIA-222-C Results for K.C. Tower

Component Type	Section No.	Elevation ft.	Size	Comb. Stress	Allow. Stress	% Capacity	Pass Fail
Leg	T6	130-150	Rohn 3 STD	0.494	1.0	49.4	Pass
Diagonal	**T8**	**90-110**	**L3 x3 x10 Ga**	**1.017**	**1.0**	**101.7**	**Fail**
Horizontal	T10	50-70	Rohn 2 STD	0.497	1.0	49.7	Pass
Top Girt	T1	230-250	L1 1/4x 1 1/4x 1/8	0.027	1.0	2.7	Pass
Redund. Horz 1 Bracing	T11	30-50	Rohn 1.5 STD	0.093	1.0	9.3	Pass
Redund. Horz 2 Bracing	T12	0-30	Rohn 1.5 STD	0.232	1.0	23.2	Pass
Redund Diag 1 Bracing	T12	0-30	Rohn 2.5 STD	0.064	1.0	6.4	Pass
Redund Diag 2 Bracing	T12	0-30	Rohn 2.5 STD	0.064	1.0	6.4	Pass
Redund Hip 1 Bracing	T11	30-50	Rohn 1.5 STD	0.002	1.0	0.2	Pass
Redund Hip 2 Bracing	T12	0-30	Rohn 1.5 STD	0.003	1.0	0.3	Pass
Inner Bracing	T10	50-70	Rohn 2 STD	0.008	1.0	0.8	Pass

Table 4.2. Summary of TIA-222-F Results for K.C. Tower (ASR = 1.0)

Component Type	Section No.	Elevation ft.	Size	Comb. Stress Ratio	Allow. Stress Ratio	% Capacity	Pass Fail
Leg	T6	130-150	Rohn 3 STD	0.686	1.0	68.6	Pass
Diagonal	**T10**	**50-70**	**Rohn 2 STD**	**11.771**	**1.0**	**1177.1**	**Fail**
Horizontal	T10	50-70	Rohn 2 STD	0.609	1.0	60.9	Pass
Top Girt	T1	30-50	L1 1/4x 1 1/4x 1/8	0.027	1.0	2.7	Pass
Redund. Horz 1 Bracing	T11	30-50	Rohn 1.5 STD	0.126	1.0	12.6	Pass
Redund. Horz 2 Bracing	T12	0-30	Rohn 1.5 STD	0.309	1.0	30.9	Pass
Redund Diag 1 Bracing	T12	0-30	Rohn 2.5 STD	0.083	1.0	8.3	Pass
Redund Diag 2 Bracing	T12	0-30	Rohn 2.5 STD	0.086	1.0	8.6	Pass
Redund Hip 1 Bracing	T11	30-50	Rohn 1.5 STD	0.002	1.0	0.2	Pass
Redund Hip 2 Bracing	T12	0-30	Rohn 1.5 STD	0.003	1.0	0.3	Pass
Inner Bracing	T10	50-70	Rohn 2 STD	0.01	1.0	1.0	Pass

Table 4.3. Summary of TIA-222-F Results for K.C. Tower (ASR = 1.333)

Component Type	Section No.	Elevation ft.	Size	Comb. Stress Ratio	Allow. Stress Ratio	% Capacity	Pass Fail
Leg	T6	130-150	Rohn 3 STD	0.670	1.333	51.1	Pass
Diagonal	**T8**	**90-110**	**Rohn 2 STD**	1.110	1.333	**88.9**	**Pass**
Horizontal	T10	50-70	Rohn 2 STD	0.589	1.333	44.3	Pass
Top Girt	T1	230-250	L1 1/4x 1 1/4x 1/8	0.020	1.000	2.7	Pass
Redund. Horz 1 Bracing	T11	30-50	Rohn 1.5 STD	0.125	1.333	9.4	Pass
Redund. Horz 2 Bracing	T12	0-30	Rohn 1.5 STD	0.308	1.333	23.1	Pass
Redund Diag 1 Bracing	T12	0-30	Rohn 2.5 STD	0.085	1.333	6.4	Pass
Redund Diag 2 Bracing	T12	0-30	Rohn 2.5 STD	0.085	1.333	6.4	Pass
Redund Hip 1 Bracing	T11	30-50	Rohn 1.5 STD	0.003	1.333	0.2	Pass
Redund Hip 2 Bracing	T12	0-30	Rohn 1.5 STD	0.003	1.333	0.2	Pass
Inner Bracing	T10	50-70	Rohn 2 STD	0.009	1.333	0.7	Pass

4.4 Taum Sauk Tower Analysis

After the Taum Sauk tower was modeled, it was analyzed using the TIA-222-F standard. For the stress checks in each member the ratios of the actual versus allowable loads and pressures were used. The equation that follows was used for the TIA-222-F checks.

$$\frac{Combined \quad Stress \quad Ratio}{Allowable \quad Stress \quad Ratio} \leq 1.333 \qquad (4.4)$$

From this check, a critical member can be selected for each type of component. The sum of the actual combined stress ratios can be divided by the allowable stress ratios to determine the percent (%) capacity of each section. This capacity is what determines if the section and eventually the tower passes or fails. Following is Table 4.4 showing the critical components in the tower and their capacities.

For towers analyzed using TIA-222-F, the code permits the engineer to increase the allowable stress by 1/3 for the load combinations if the tower is less than 700 feet tall. This increase in allowable stress is stated in the TIA-222-F, section 3.1.1.1. This makes the stress ratio become 1.333, instead of 1.0. When the Taum Sauk tower is analyzed for the F standard with a stress ratio of 1.333 the tower receives a rating, based on the most critical tower component, of 81.7% which indicates that the tower passes. The critical components for the tower with a stress ratio of 1.333 can be seen in Table 4.4 below. Although the tower passes, the drawings were incomplete regarding attachments to the tower and feedline information. The tower was analyzed using assumed attachments and feedlines based on the drawings. A more complete analysis is recommended and could be done if actual attachments and their placements were known.

Table 4.4. Summary of Wind/Ice Loading Results for Taum Sauk Tower using TIA-222-F

Component Type	**Section No.**	**Elevation ft.**	**Size**	**Comb. Stress Ratio**	**Allow. Stress Ratio**	**% Capacity**	**Pass Fail**
Leg	T13	0-10	P1.25x.078125 in	0.825	1.333	61.9	Pass
Diagonal	T5	100-110	7/16 in	0.933	1.333	70	Pass
K-Brace	T14	10-20	7/16 in	0.163	1.333	12.2	Pass
Horizontal	T14	0-10	7/16 in	0.065	1.333	4.9	Pass
Top Girt	T15	0-10	2 x 1/2 (inches)	0.167	1.333	12.5	Pass
Guy A	**T2**	**140**	**3/8 in.**	**1.089**	**1.333**	**81.7**	**Pass**
Guy B	T2	140	3/8 in.	0.821	1.333	61.6	Pass
Guy C	T2	140	3/8 in.	1.006	1.333	75.5	Pass
Top Guy Pull-off	T13	20-30	1 1/4 in	0.116	1.333	8.7	Pass

4.5 Summary of Results

The Kansas City and Taum Sauk towers were analyzed for wind and ice loadings using ERITower. It was found that both towers passed using the F version of the TIA-222 tower code. In both cases the diagonal bracing on the tower controlled the stability of the tower.

4.5.1 Kansas City Tower Results

The Kansas City tower was analyzed using the C and F versions of the TIA-222 code. The tower was built using requirements from the C version of the code, however it was found that the tower failed due to the tower capacity being 101.7%. The diagonal bracing at elevations of 90 to 100 feet controlled the tower stability. Other members in the tower were loaded to 50% of their capacities(see Table 4.1).

Using the F version of the code the tower passes. The code calls for an allowable stress ratio, ASR, of 1.0 for most towers, however for shorter towers an ASR of 1.333 can be used. The Kansas City tower is allowed to use the ASR of 1.333 and passes at 88.9% capacity. The diagonals again are the controlling members in this tower, this time at elevations from 90 to 110 feet. Other members in the tower are only loaded to at most 51.1% of their capacity (see Table 4.3).

4.5.2 Taum Sauk Tower Results

Taum Sauk tower was analyzed only using the F version of the TIA-222. Since the tower is only 150 feet tall, less than 700, an allowable stress ratio of 1.333 was used. The guys connected to the tower controlled total stability of the tower. The guy that controlled was loaded to 81.7% capacity. The diagonals of the actual structure were loaded to 70% capacity. All other members types were stressed to less than 70% capacity, therefore the tower passed under wind and ice loading conditions outlined in the TIA-222 F.

5 Analysis Under Seismic Loading

5.1 Introduction

The Taum Sauk Tower is located in southern Missouri near the New Madrid fault line. This area is very seismically active, therefore a seismic analysis is also needed to determine the stability of the tower. The seismic analysis was done using SAP *v9.0*.

SAP is a static and dynamic structural analysis program that includes linear and non-linear analysis capabilities. Seismic analysis can be performed using SAP and the ground motion can be modeled using spectrum or time history functions. Of particular interest for this project were the dynamic modeling capabilities, which can be performed using response spectrum analysis, time history analysis, and combinations of loading scenarios. Modal analysis was performed using Eigenvector analysis for response spectrum function and Ritz vector for time history function. SAP allows the user to input the response spectrum function. Preprocessing in SAP utilizes a graphical interface for defining the tower geometry and properties of members and for defining loads and load combinations. Post processing provides output for internal forces and moments, displacements, mode shapes, and design checks.

5.2 Taum Sauk Model Details

The tower was modeled using the information provided by structural tower drawings (Appendix A). Leg, diagonal, and cable sizes were used as provided in the drawings. Select element sizes were verified by field measurements. Tower attachments as shown in the drawings were included in the model. The attachments on the existing tower (such as additional antennas) were not modeled in this analysis, since no information was available at the time. Design drawings were modeled by SAP to predict the response under earthquake loads.

The tower was modeled as a frame structure made up of 15 sections, each 10 ft in length. The legs of the tower were modeled as tube elements and the diagonals were modeled as solid bars. Guy cables were modeled as solid bars with moment releases added at the ends to simulate tension-only cables. The use of bar members was selected to overcome limitations in the ability of SAP to model tension-only members. Results, therefore, indicate compressive forces in the guy cables, which should be disregarded. Tensions in the cables were modeled as external forces applied on the tower at the location where the cables meet the tower legs along the direction of he cables. The two antennas shown on the drawings were modeled as solid rod elements, and attached rigidly to the tower. The SAP model for the Taum Sauk tower schematic is shown in Figure 5.2.

5.3 Taum Sauk Tower Loading

The dead load (weight) of the tower was calculated automatically in SAP based on the material properties specified, and was included in the dynamic analysis. The tensions in the cables were modeled as external forces applied on the tower at the location where the cables meet the tower leg and in the direction of cables.

Response spectrum functions shown in Figures 5.3 and 5.4 were specified for two different earthquake seismic events. The standard spectrum function of Figure 5.3 was used to produce a normalized standard spectrum with respect to gravitational acceleration as shown in Figure 5.5.

The response spectrum function was applied in three directions: X, Y and Z (vertical) directions as shown in Figure 5.6. The time history functions in Figure 5.7 were used for the nonlinear time history analysis based on 16 modes from Ritz vector modal analysis.

Two load combinations were used in the SAP modal analysis:

- The first combines the dead load and cable tension with 100% spectrum function in the X-direction, 30% spectrum function in the Y-direction, and 30% spectrum in the Z-direction.
- The second combines the dead load, cable tension, and 100% time history function in the X-direction.

SAP was used to perform the dynamic analysis for these two combinations, and the results in terms of nodal displacements, mode shapes, and internal member forces and moments, were collected, and evaluated. Summary of the results is provided next.

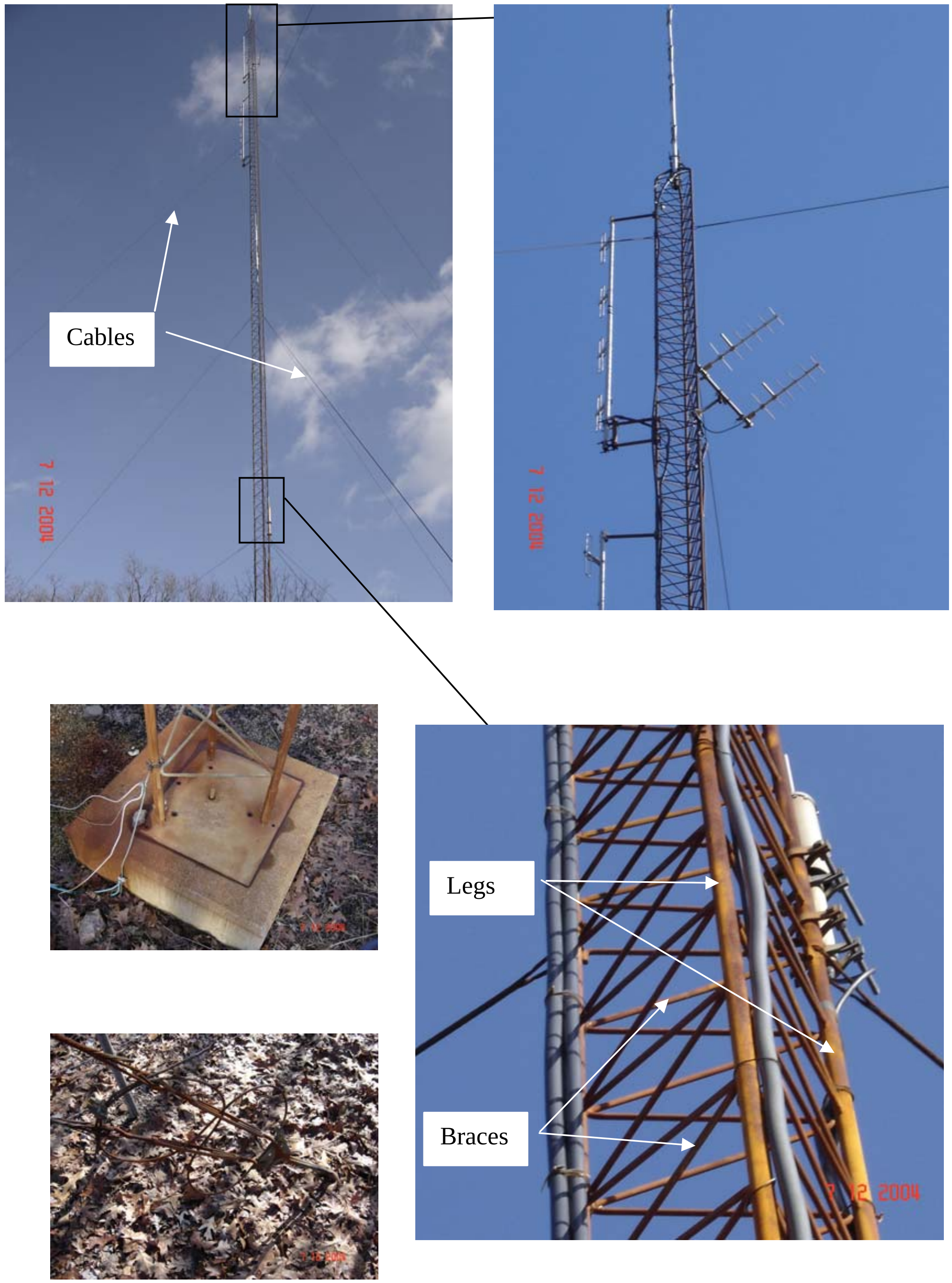


Figure 5.1. Key Elements of Taum Sauk Tower

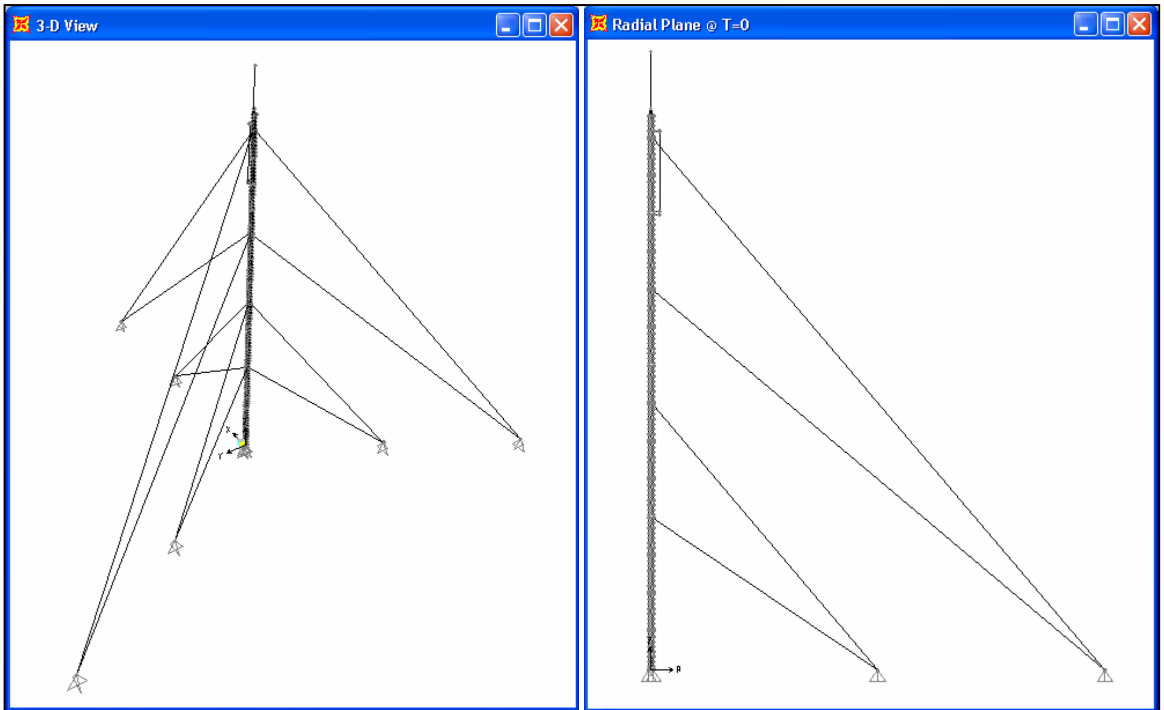


Figure 5.2. SAP model of Taum Sauk tower

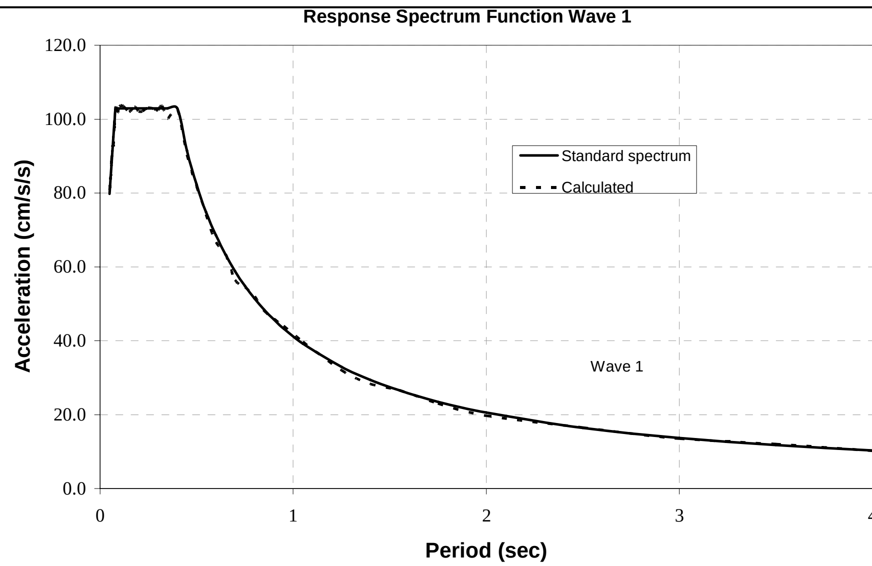


Figure 5.3. Response spectrum function for wave 1

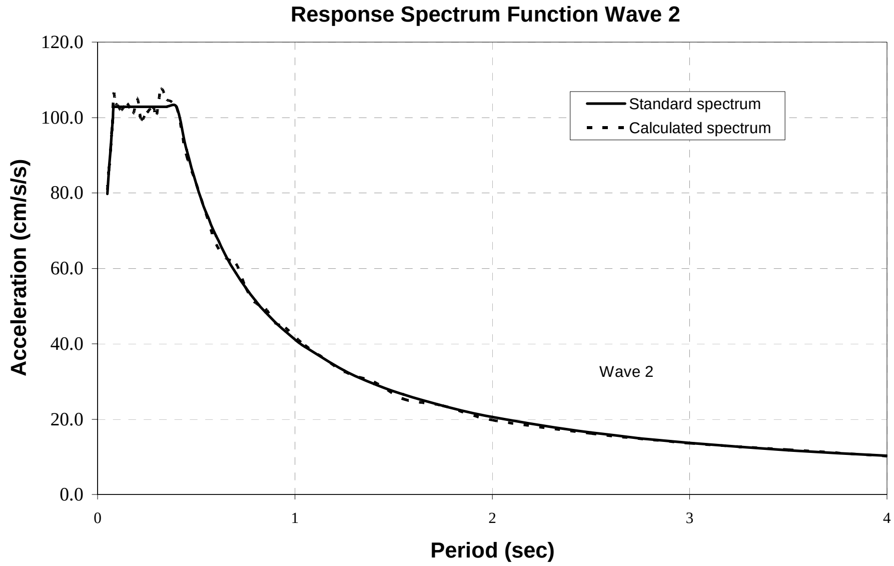


Figure 5.4. Response spectrum function for wave 2

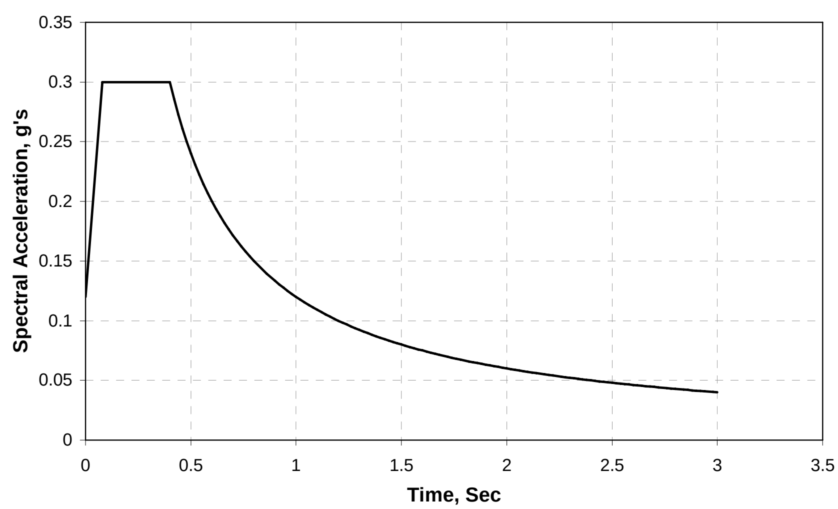


Figure 5.5. Standard Response Spectrum Function

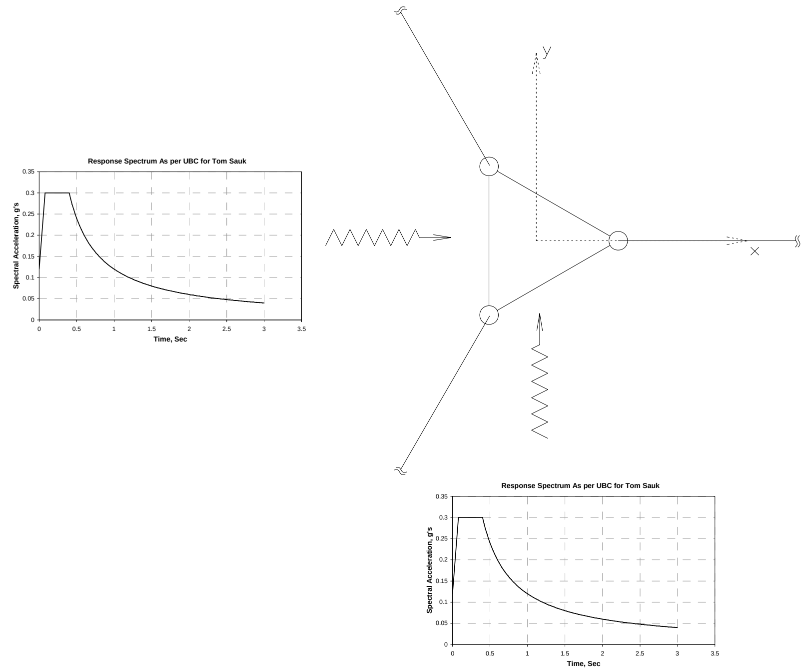


Figure 5.6. Load of Response Spectrum Function on the Taum Sauk Tower

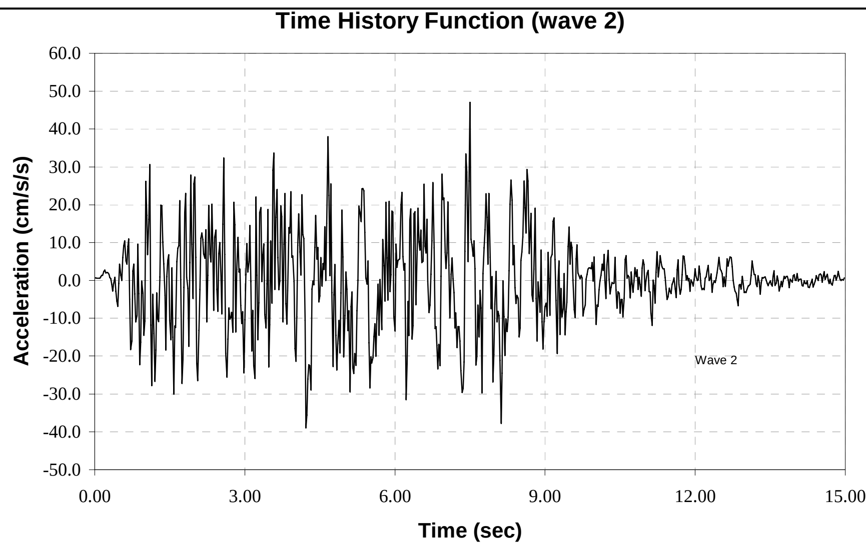


Figure 5.7. Time History Function (wave 2)

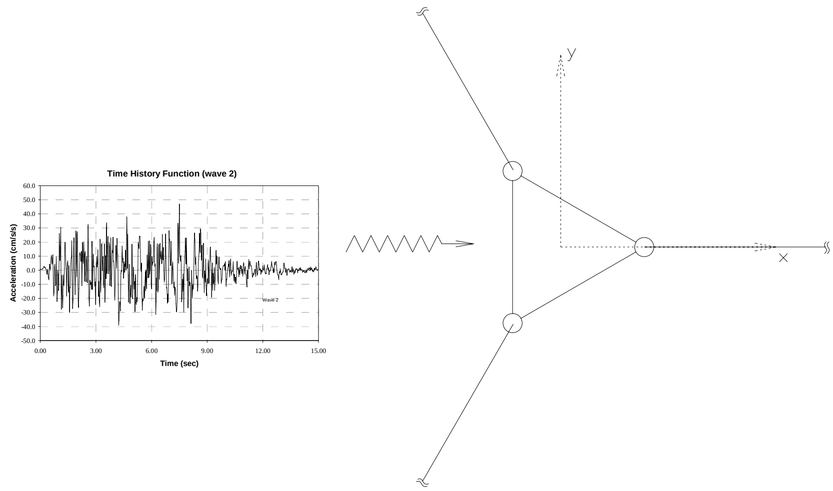


Figure 5.8. Loading of the time history functıon on the tower

5.4 Results from Seismic Analysis

<u>Mode shapes</u>

A total of sixteen mode shapes were selected for the analysis. Selected modes are shown in Figure 5.9 and the remaining 16 modes are shown in Appendix B. The modal analysis was used for the dynamic analysis using the response spectrum loading function as described earlier.

<u>Defining Critical Members</u>

Detailed evaluation of the results of the SAP analysis revealed that the braces were stressed the most when compared to the leg and cable members. The critical braces were defined using the calculated brace axial forces from modal analysis for the standard spectrum function, modal analysis for the time history function, spectrum function combination, and time history function combination for maximum and minimum values.

Axial forces for all sixteen modes are summarized in Appendix B. The maximum results from all combinations are shown in Figure 5.10, which indicates the location of the most critical braces in the tower are at or near locations where the guys connect to the tower legs. These critical braces were evaluated further in the parametric study described later in this section of the report.

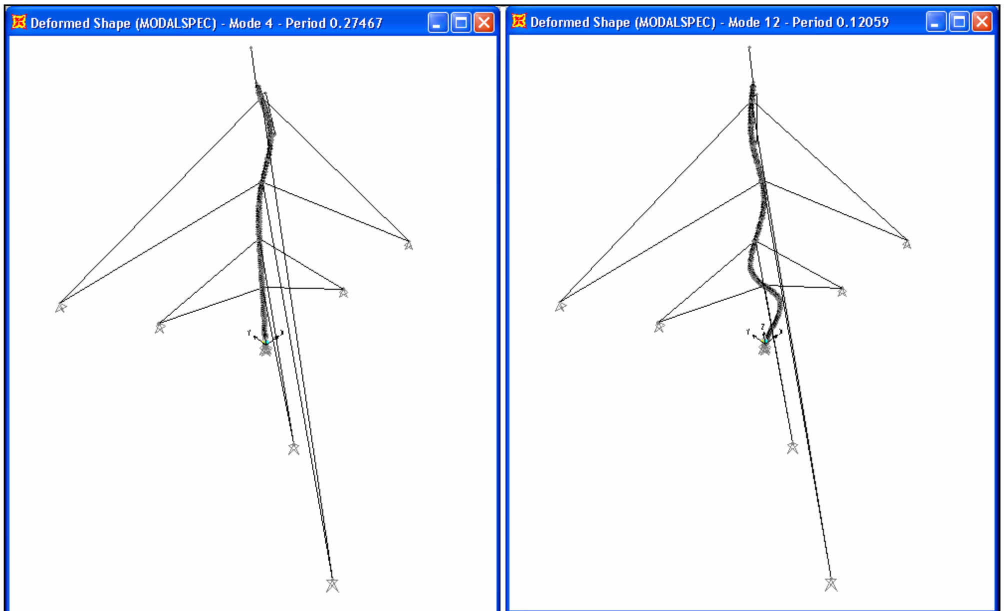


Figure 5.9. Selected mode shapes 4 and 12

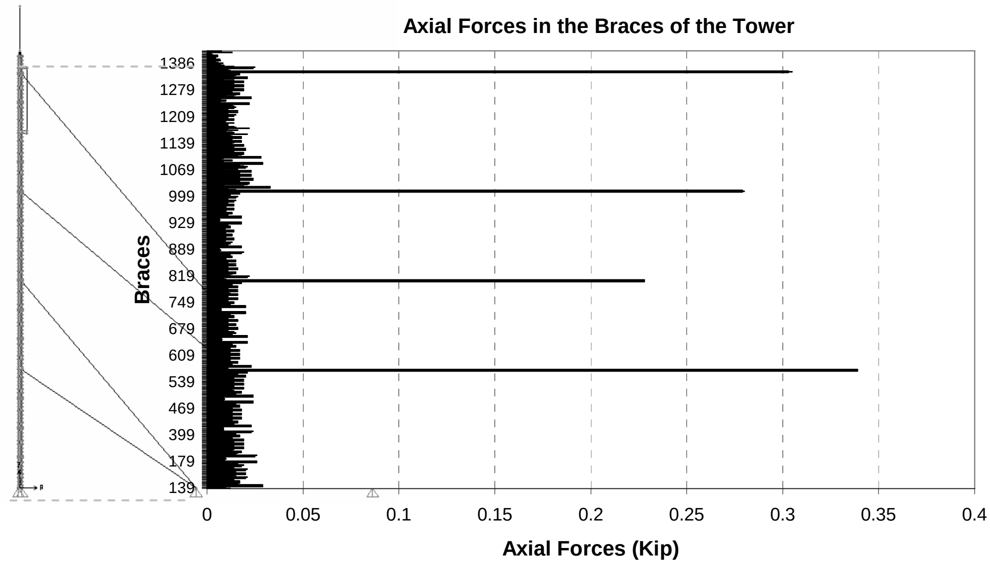


Figure 5.10. Axial Forces of Braces for Undamaged Structure of the

5.5 Summary

Dynamic modeling was only completed for the Taum Sauk tower because it is in a region of high seismic activity. The Kansas City tower is not in a very seismic area, therefore dynamic analysis was not needed at this time. SAP2000 v.9 was used to model and analyze the tower. Time history and response spectrum functions were obtained from Tulasi (2005) and used to apply loads to the tower. Once analyzed, it was shown that diagonal bracing controlled the stability of the tower. Specifically the bracing near and around the areas where guys are attached to the tower showed the maximum axial member forces. These high axial forces gave locations of the critical braces in the tower. These critical braces are further analyzed in a parametric study of the tower in the next chapter.

6 Parametric Evaluation

6.1 Introduction

The Missouri Department of Transportation's radio tower network was installed in the 1950's and 1960's as a state wide communication system for civil defense. These towers are now around 40 to 50 years in age and are considered to be old. As time goes by, the towers tend to deteriorate slowly, as would be expected of any structure subjected to external influences and the environment. Recently, a systematic way of categorizing the towers was developed to help determine their physical condition and assess their vulnerability. This system creates a condition index based on the principal components of the tower and the environmental conditions to which the tower is subjected.

A series of parametric studies was conducted to evaluate the effects of simulated damage (age-related deterioration) to the Taum Sauk tower and its associated response under seismic, wind and ice loading. This information can be tied back into the condition indexing system. These studies help understand the effects of rust damage, in terms of cross-sectional area, to the principal components of the tower. The wind and ice loading analyses using ERITower software is presented first followed by the seismic loading results using SAP software.

6.2 Condition Indexing System

A condition indexing (CI) system is a methodology used to systematically quantify a structure's physical condition. The CI ranges from 0 to 100 with 0 being the worst possible condition and 100 being the best. CI systems are very valuable tools for complex, networked facilities such as the MODOT communication tower system. To help maintain the structural integrity of the tower system, a function-based condition indexing system for guyed communication towers was developed (Tulasi, 2005) and is summarized in this section. This system has a series of steps used to eventually determine how a tower will react in an emergency situation and also prioritize the towers in terms of need of repair.

Step 1 of the indexing system is to identify the functional components of the system. For guyed towers, there are four basic components: the guys, central mast, foundation and environmental loading. Figure 6.1 shows the four functional components of the system.

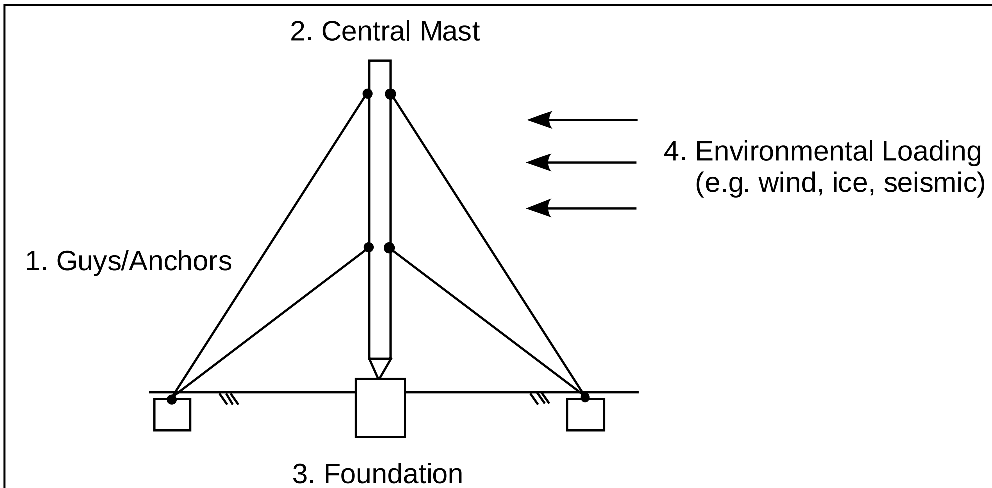


Figure 6.1 Functional components of guyed radio towers

Once the components have been determined it is then necessary to create an interaction matrix. This matrix will identify how each of the components affects the other components and the structure as a whole. The basic principle of this component matrix can b seen in Figure 6.2.

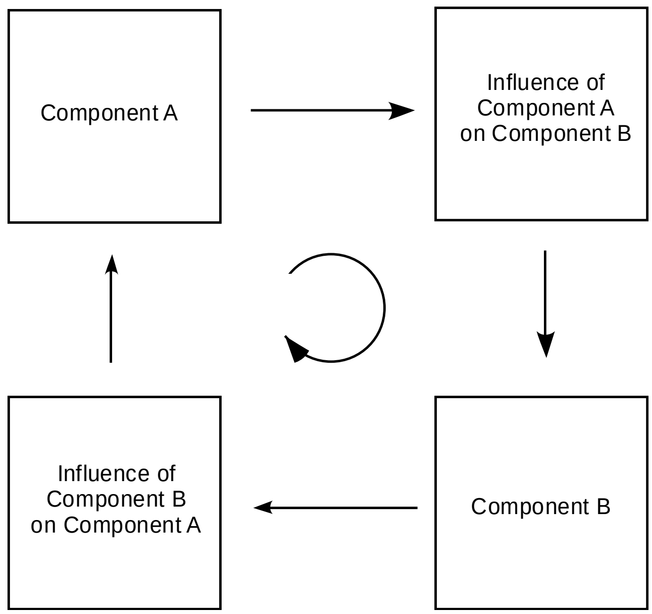


Figure 6.2 Four component interaction matrix

Step 3 and 4 are in-depth steps using a 7x7 indexing matrix that was created for the guyed radio towers. Step 3 codes the interaction matrix to determine the importance of each interaction in the matrix. Step 4 uses these importance factors to determine a range between the ideal and failed conditions for each component. For this step each component of a single tower is ranked from 0 to 100 based on ideal conditions determined.

Step 5 is where a formula is determined to find the condition index of the tower. A detailed description of how the weighting factors are determined can be found in “Function-Based Condition Indexing For Guyed Communication Towers”, by Devi

Tulasi (2005). Below are the results of that paper and the equation to determine the actual condition index of a tower.

Table 6.1 Condition Indexing Weight factors

Subunit	Cause Score	Effect Score	Total Score	Weight	Weight Factor
Guy Cables	6	10	16	16/52	0.31
Guy Anchors	2	8	10	10/52	0.19
Central Mast	5	11	16	16/52	0.31
Foundation	3	7	10	10/52	0.19
Total	16	36	52	---	---

The following equation is used to determine the CI of the tower,

$$\boldsymbol{CI = CI_{gc}(0.31) + CI_{ga}(0.19) + CI_{cm}(0.31) + CI_{fd}(0.19) = 0 - 100} \qquad (6.1)$$

where CI_{gc}, CI_{ga}, CI_{cm} and CI_{fd} are the condition indicies for the guy cables, guy anchors, central mast and the foundation, respectively.

Once the condition index has been determined the tower can be prioritized with respect to REMR(Repair, Evaluation, Maintenance, Rehabilitation) and resources of the department of transportation can be allocated as needed. Table 6.2 shows the range of CI’s and recommended action for each index (Tulasi, 2005).

Table 6.2 Condition indexing scale for guyed towers

Condition Index	Condition Description	Recommended Action
85 – 100	Excellent: No noticeable defects; some ageing or wear may be visible	Immediate action is not warranted
70 – 84	Very Good: Only minor deterioration or defects are evident	
55 – 69	Good: Some deterioration or defects but function is not significantly affected.	Economic analysis of repair alternatives is recommended to determine appropriate action
40 – 54	Fair: Moderate deterioration but function is adequate	
25 – 39	Poor: Serious deterioration and function is inadequate	Detailed evaluation is required to determine the need for repair, rehabilitation, or reconstruction. Safety evaluation is recommended.
10 – 24	Very Poor: Extensive deterioration; barely functional	
0 – 9	Failed: No longer functional	

The dynamic/structural analysis was performed to help rationalize the creation of the CI interaction matrix, which is used to determine the CI for the towers. Following is a parametric study of the Taum Sauk tower in which the members of the tower were modeled as if they had been damaged 0 to 50 percent. This helps to understand what members are the most critical to repair and how damaging one part of the tower can affect other components. Therefore using this part of the research will allow the developers of the condition indexing system to better weigh and rank the components of the tower.

6.3 Wind and Ice Loading

The Taum Sauk tower was first evaluated using ERITower software under wind and ice loading for various assumed damage (deterioration) levels. Damage levels were assumed to range from 0%, or no damage, to 50% damage. Simulated deterioration was introduced to various components of the tower, namely the legs, braces, and guys.

6.3.1 Effects of Guy Damage

Simulated deterioration was introduced to the guys by decreasing the cross sectional area of the guys from 0% to 50% by 10% increments. The tower was loaded as mentioned previously in Chapter 4 for all cases of deterioration. Table 6.3 and Figure 6.3 show how the guys, legs and bracing reacted to the damage of the guys. As expected, as the percent damage to the guys is increased, the guys percent capacity used also increases. It is also seen that the guys control the tower rating when damage is introduced in the guys only.

The legs and bracing react similarly to the damaging of the guys. As guy damage increases, the capacities of the legs and braces are somewhat relieved. In other words, the braces and legs are at a lower percentage of their capacities when there is more damage to the guys. This is because when the guys are at 0% damage, the points near the connections of the guys to the towers are basically rigid points. Therefore, the critical members are at or around the connection points and are highly stressed, due to the fact that they have minimal displacement or movement. As the guys deteriorate, the points of connection are able to move slightly, thus relieving some of the forces in the members. In Figure 6.3, the graph depicting Guy Damage (%) vs. Leg Capacity (%) increases when the damage to the guys is increased from 40% to 50%. This is because at this point the critical leg member changes. The previous damage percentages, 0% to 40%, had a different critical member and when 50% damage was reached, a new member in the same section of the tower was stressed a little more causing the jump in the graph.

Table 6.3. Guy Damage Effect on Tower

Guy Damage %	Tower Rating %	Guy Capacity %	Leg Capacity %	Bracing Capacity %
0	81.7	81.7	61.9	70.0
10	87.7	87.7	59.8	69.7
20	95.1	95.1	58.0	69.0
30	104.6	104.6	56.7	68.2
40	117.2	117.2	56.3	67.3
50	134.8	134.8	58.1	66.5

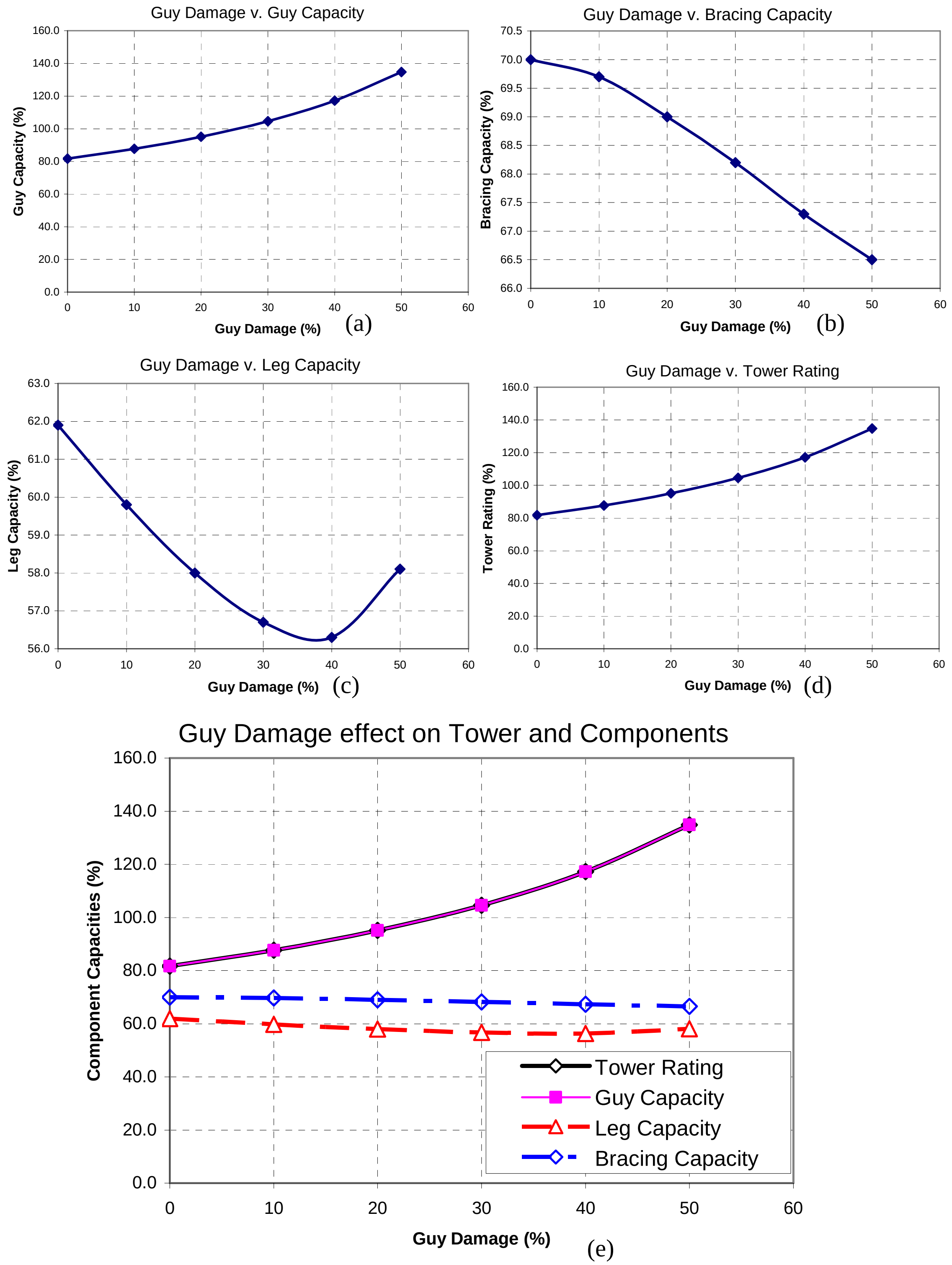


Figure 6.3. Effect of Guy Damage on Tower Rating and Components Capacities

6.3.2 Effects of Bracing Damage

Simulated deterioration was introduced to the bracing by decreasing the cross sectional area of the guys from 0% to 50% by 10% increments. The tower was loaded as mentioned previously in Chapter 4 for all cases of deterioration. Tables 6.4a, 6.4b and Figure 6.4 show how the guys, legs and bracing reacted to the damage of the braces. It can be seen in Figure 6.4(d) that the braces reach 100% capacity before the braces are damaged 10%. This shows that the braces are critical to the structural integrity of the tower. Figure 6.4(c) and Table 6.4(b) shows the effects of damaging the braces on the braces from 0% to 10%. It is shown that the braces reach 100% capacity at about 7.5% damage to the braces. This controls the tower rating since the guys and legs percent capacities decrease and never reach 100%.

The guys and legs were affected similarly due to the percent damage increase of the bracing. As the brace damage was increased the guy and leg percent capacities were relieved by 1.1% and 3.5%, respectively. These are very small decreases in capacities, which are believed to be related to the loading of the tower. As the cross sections of the braces are decreased, the area to which the wind and ice loadings are applied decreases, therefore making the total load on the tower slightly decrease. In reality, there would not be much decrease in the area to which the load is applied since rust coats the member. There would be a decrease in area giving strength to the tower, as rust is very weak, but the area to which loads are applied would not decrease. For this research the cross sections are decreased as it is a simple way to account for deterioration.

Table 6.4a. Bracing Damage Effect (0 – 50%)

Bracing Damage %	Tower Rating %	Guy Capacity %	Leg Capacity %	Bracing Capacity %
0	81.7	81.7	61.9	70.0
10	484.0	81.5	61.0	484.0
20	2783.5	81.2	60.1	2783.5
30	469.6	81.0	59.2	469.6
40	2738.8	80.8	58.7	2738.8
50	3361.3	80.6	58.4	3361.3

Table 6.4b. Bracing Damage Effect (0 – 9%)

Bracing Damage %	Tower Rating %	Guy Capacity %	Leg Capacity %	Bracing Capacity %
0	81.7	81.7	61.9	70.0
5	87.3	81.6	61.5	87.3
6	92.1	81.5	61.4	92.1
7	98.0	81.5	61.3	98.0
8	106.5	82.5	61.2	106.5
9	123.9	81.5	61.1	123.9

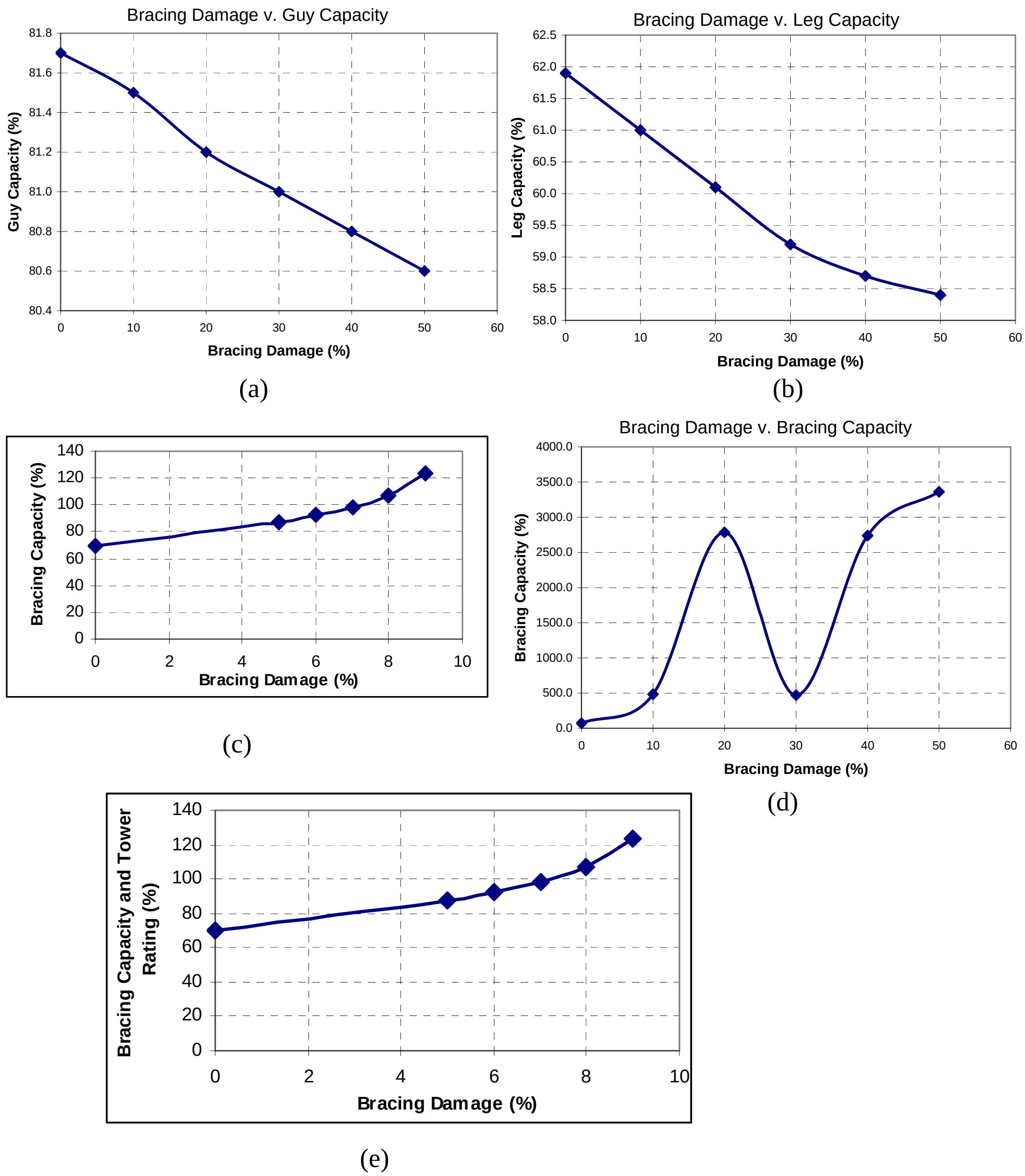


Figure 6.4. Effect of Braces Damage on Tower Rating and Components Capacities

6.3.3 Effects of Leg Damage

Simulated deterioration was introduced to the legs by decreasing the cross sectional area of the legs from 0% to 50% by 10% increments. The tower was loaded as mentioned previously in Chapter 4 for all cases of deterioration. Table 6.5 and Figure 6.5 show how the guys, legs and bracing reacted to the damage of the legs. Figure 6.5 (c) shows that as damage is introduced to the legs, the percent capacity used of the legs increases. The tower rating for leg damage is shown in Figure 6.5 (d). This graph is a compilation of the bracing, guy and leg capacities. However the tower fails when the legs reach 100% capacity, making the tower controlled by leg capacity when the legs are damaged. The tower fails when the legs are damaged 42%. The bracing and guy capacities never reach 100% when the legs are damaged to a maximum of 50%.

The braces are not affected much by the damage to the legs. The percent capacity used for the braces increases from 70% to 72.6% a difference of 2.6%. The percent guy capacity decreases as the legs are damaged, as they did when the braces were damaged in the previous section. This 1.2% decrease in percent guy capacity used can again be attributed to the decrease in area to which the ice and wind loads are applied.

Table 6.5. Leg Damage Effect on Tower

Leg Damage %	Tower Rating %	Guy Capacity %	Leg Capacity %	Bracing Capacity %
0	81.7	81.7	61.9	70.0
10	81.5	81.5	67.9	70.3
20	81.4	81.4	75.4	70.6
30	85.0	81.1	85.0	71.0
40	97.6	80.8	97.6	71.6
50	116.4	80.5	116.4	72.6

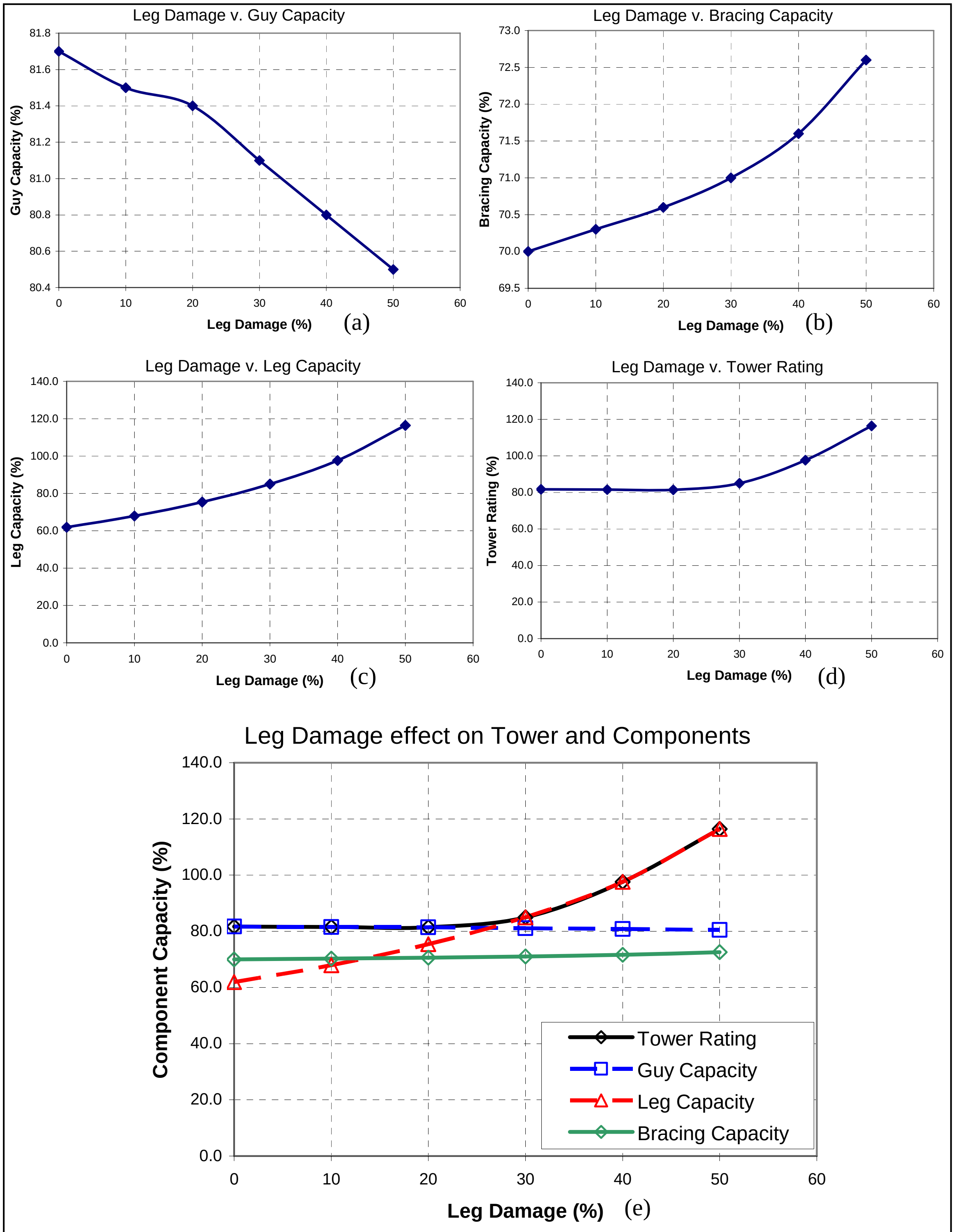


Figure 6.5. Effect of Leg Damage on Tower Rating and Components Capacities

6.4 Seismic Loading

The Taum Sauk tower was also evaluated using SAP software under seismic loading for various assumed damage levels. The damage levels were assumed to range from 0%, or no damage, to 50% damage. Damage was introduced to main components of the tower, namely the legs, braces, and guys. Damage was also introduced to all tower components at the same time and the effects on the member forces were evaluated.

To evaluate the effect of damage on the tower members, such as the result of corrosion, a parametric study was conducted using SAP. The parameters varied in this section are the member sizes of the legs, braces and cables. Damage was simulated by reducing the cross sectional area of the members from 0% to 50% at 10% increments. Four different cases of damage were considered (Table 6.6). In the first case, the damage was only introduced on the tower legs, but keeping the rest of the members undamaged. In the second case, the damage was only introduced on the tower braces, but keeping the rest of the members undamaged. In the third case, the damage was only introduced on the tower cables, and keeping the rest of the members undamaged. In the fourth case, the damage was introduced on all tower members.

Table 6.6 Tower member sizes for parametric study

BRACES		d=	0.365	ft		
%damage	100	90	80	70	60	50
Diameter (d)	0.365	0.3285	0.292	0.2555	0.219	0.1825

COLUMN		d=	1.42	ft	t=	0.0059 ft
%	100	90	80	70	60	50
Wall thickness (t)	0.0059	0.00531	0.00472	0.00413	0.00354	0.00295

CABLES		d1=	0.0313	ft	d2=	0.026 ft
%	100	90	80	70	60	50
Diameter (d1)	0.0313	0.02817	0.02504	0.02191	0.01878	0.01565

6.4.1 Effects of Component Damage on Critical Brace

The member forces in the critical braces identified in Chapter 5 were investigated in the parametric study (Figure 6.6). The most critical brace member # 556, which is located near the lower guy location on the tower, was further investigated in this parametric study. The axial force history for this brace is shown in Figure 6.7. Variation of axial force in the critical brace member # 556 with the variation in damage to individual components of the tower as well as to all members of the tower is shown in Figures 6.8 through 6.11

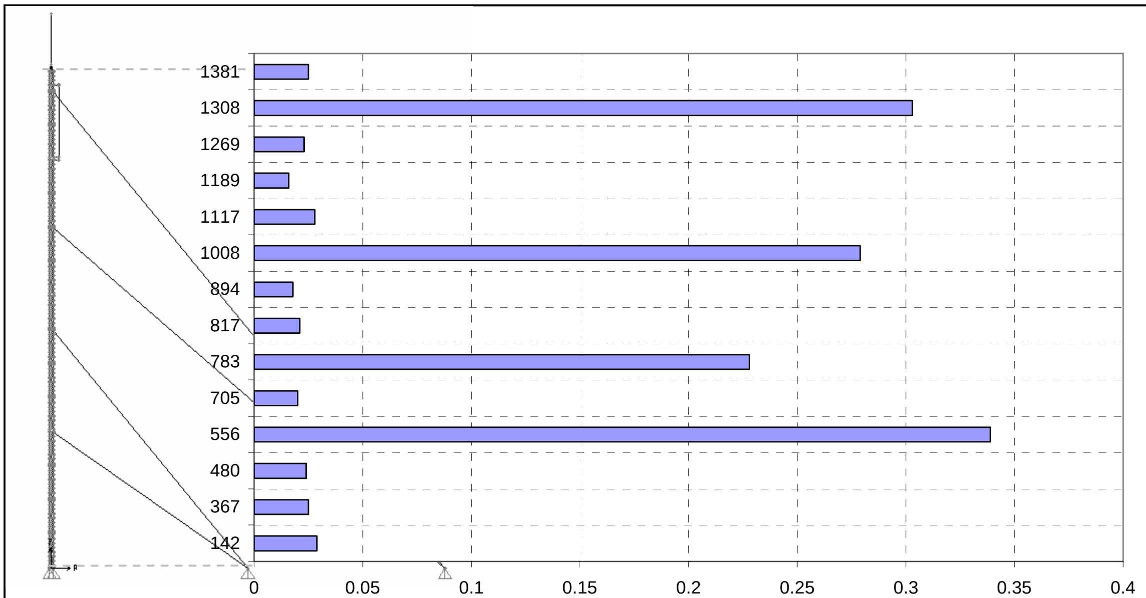


Figure 6.6. Axial Forces in Selected Critical Braces of the Tower

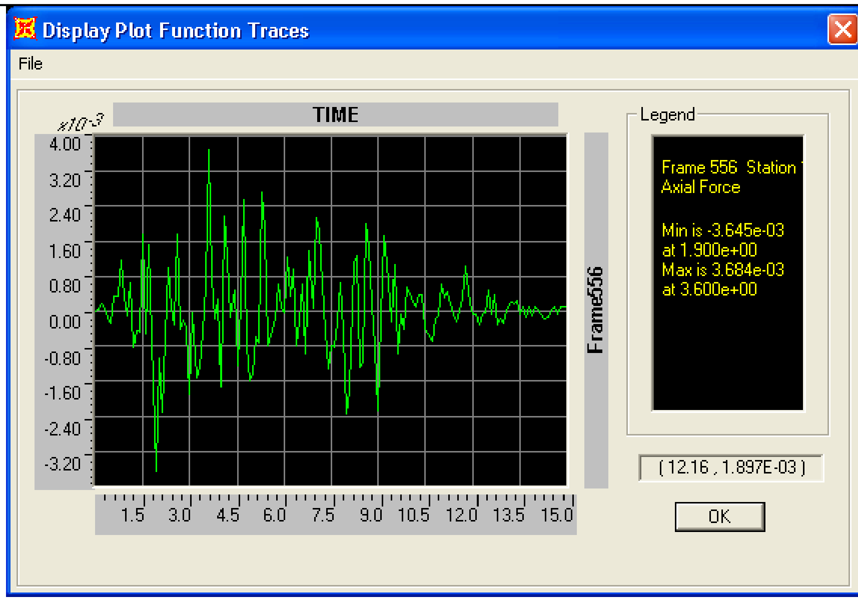


Figure 6.7. Axial Force in kips with Time at Selected Brace 556 for Time History Function

Figures 6.8 through 6.11 show the effects of damage to structural members on the critical brace member # 556. It is shown in Figures 6.8 and 6.10 that when damage is introduced to the braces or to the tower legs, the axial forces in the brace #556 decreases. Figure 6.9 shows as the guy cables are damaged the axial force in the braces increases. Knowing how the braces react to damage in the other members of the tower shows that the guys are critical to the structural integrity of the tower. Figure 6.11 also shows that as all members of the tower are damaged, the axial forces in brace #556 increase. This is to be expected since the guys are still being damaged in this test.

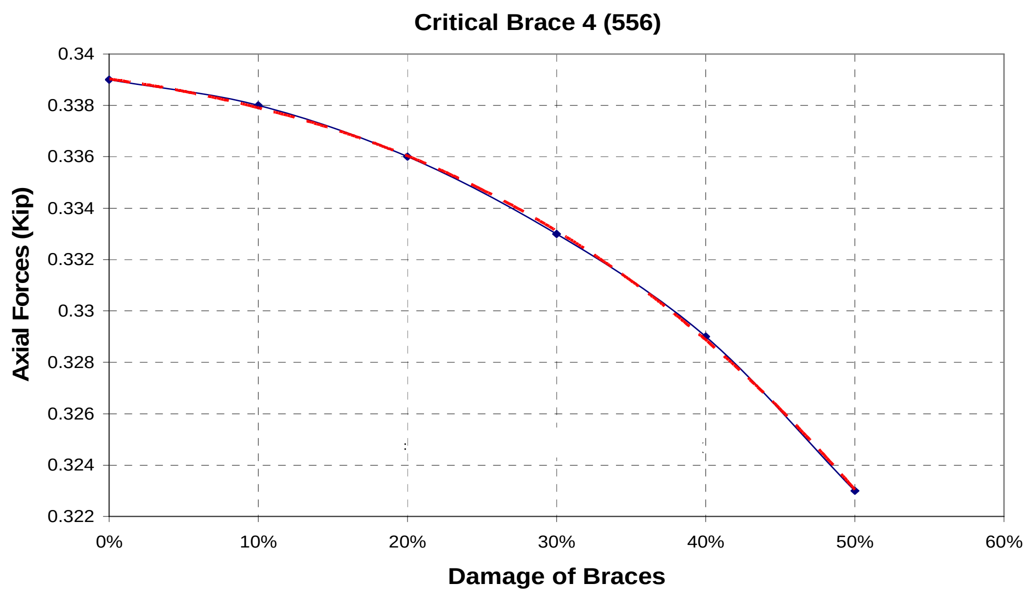


Figure 6.8. Axial Force for Critical Brace # 556 with Damage to Braces

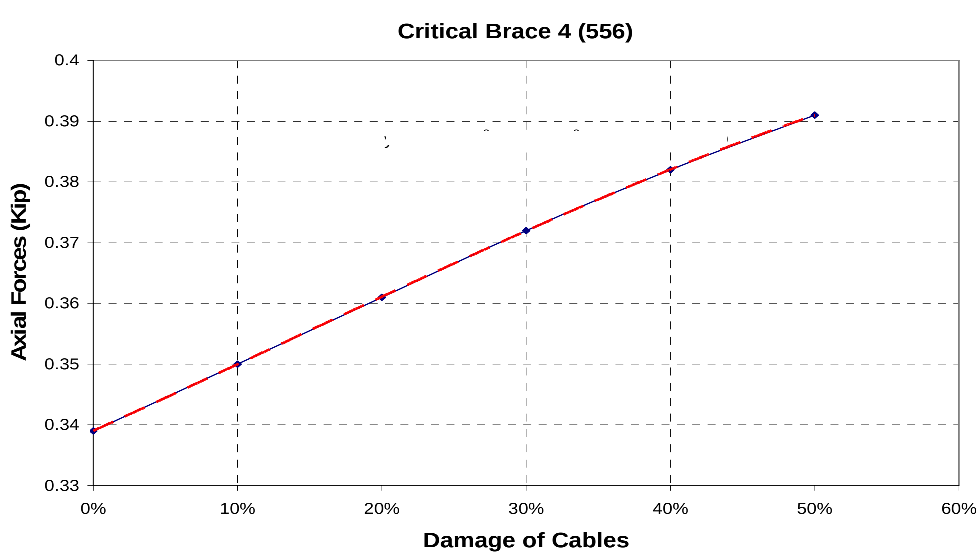


Figure 6.9. Axial Force for Critical Brace # 556 with Damage to Cables (Guys)

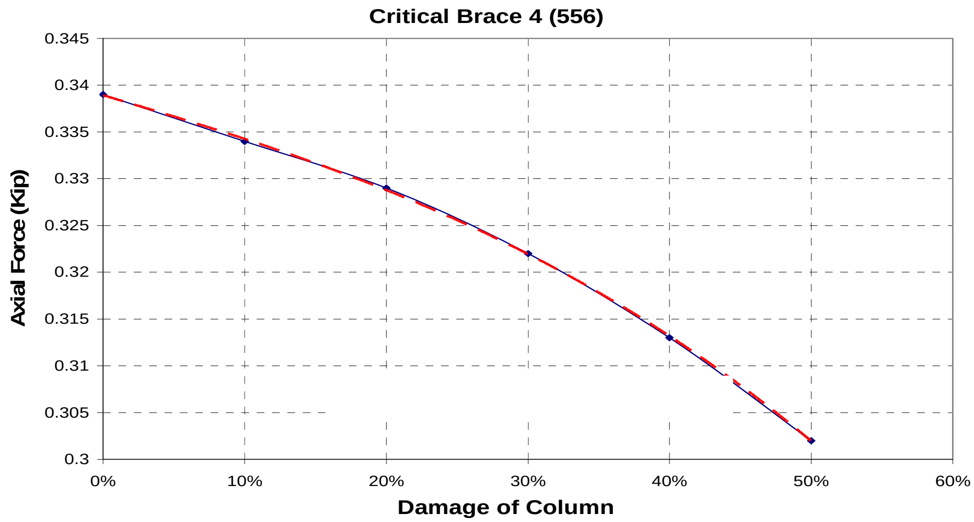


Figure 6.10. Axial Forces for Critical Brace # 556 with Damage to Legs

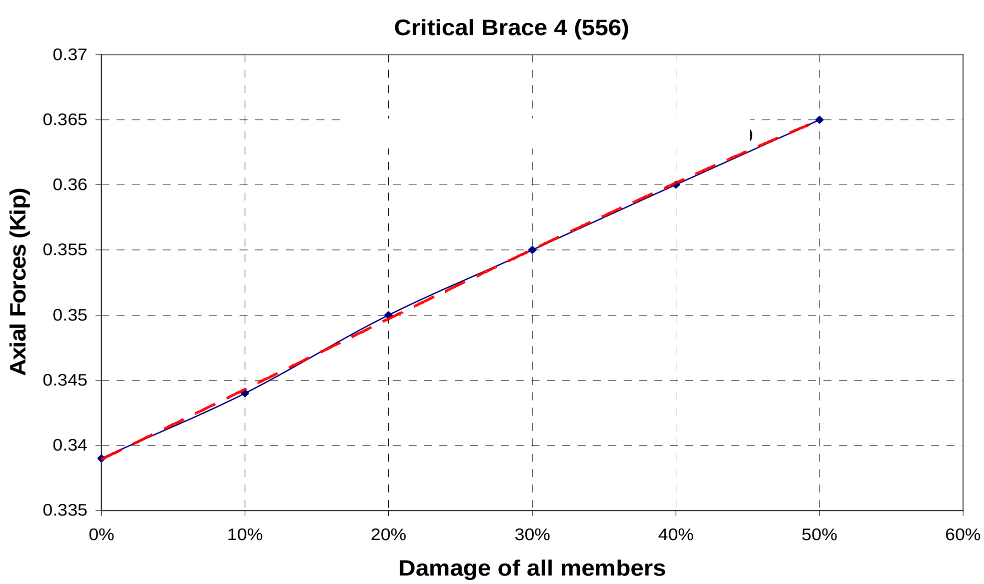


Figure 6.11. Critical Brace #556 with damage to all members

6.4.2 Effects of Component Damage on Critical Cable

Similarly, the effect of damage on the member forces in the guy cables was investigated. Critical cables were defined by evaluating the axial forces in the guys for the undamaged tower under seismic loading (Figure 6.12). The lowest guys were stressed the most and thus were selected for further evaluation. The axial force in the bottom guys with time is shown in Figure 6.13. As mentioned previously, the guy cables were modeled in SAP using bar elements. The apparent compressive axial forces shown in Figure 6.13 should be disregarded. Variation of axial force in the critical guy with the variation in damage to individual components of the tower as well as to all members of the tower is shown in Figures 6.14 through 6.17. The guy forces shown on these figures do not include initial guy tension prior to dynamic loading. The initial tensile force of (0.8 kips), therefore, should be added to the computed excess dynamic loading forces.

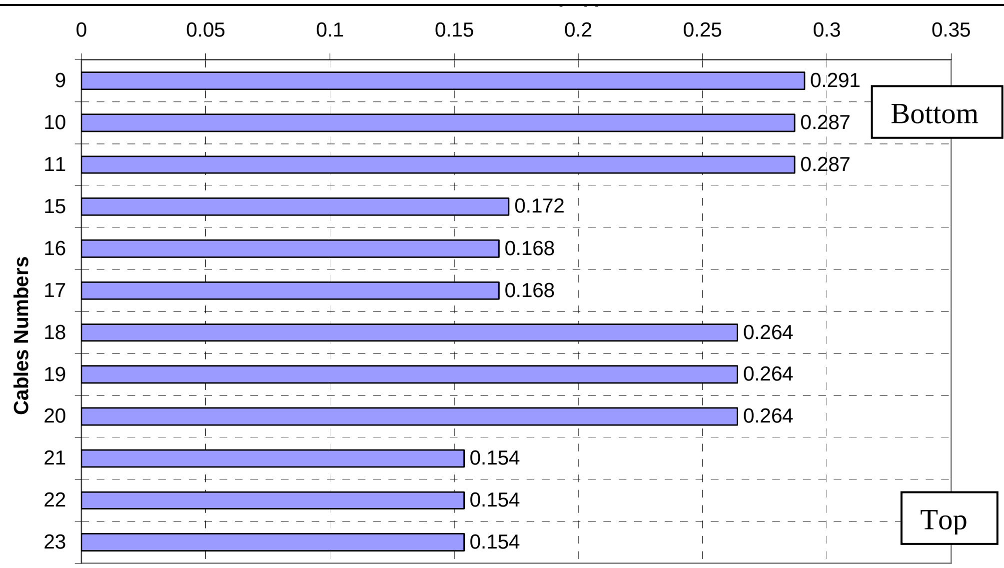


Figure 6.12. Axial Forces in Cables (kips) without any Damage in the Tower

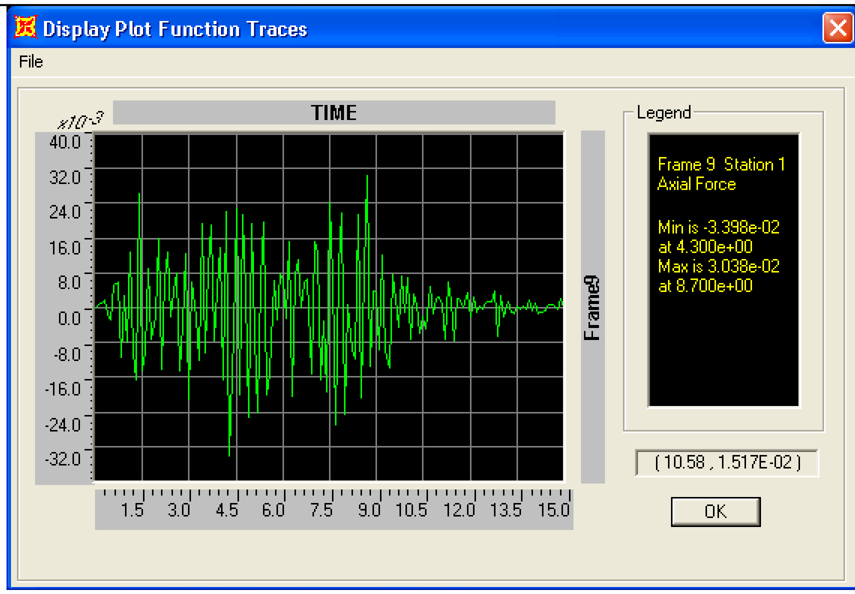


Figure 6.13. Axial Forces with Time at Selected Cable 9 for Time History Function

Figures 6.14 to 6.17 show the effects of damaging specific components of the tower on the critical guy cable 9. Figure 6.14 shows that as damage of the braces increases, the axial forces in cable 9 are stable from 0 to 20 percent and then increase 0.001 kip from 20 to 30 percent damage and finally level off. This basically shows that the damage to the braces had almost no effect on the critical guy cable 9. When damage to the cables is introduced, the axial force in cable 9 decreases as seen in Figure 6.15. As the cables are damaged some relaxation will be seen in the cables, thus decreasing the amount of axial force in each cable. Figure 6.16 shows the effect of introducing damage to the legs of the tower. As more damage is introduced to the legs, the axial force in cable 9 increases around 0.15 kip. The only other increase in axial force in the cables is approximately 0.001 kip when damage is introduced to the braces. This is fairly insignificant in comparison to the leg damage increase, which shows that the axial force carried by the cables is affected greatly by the condition of the legs of the tower.

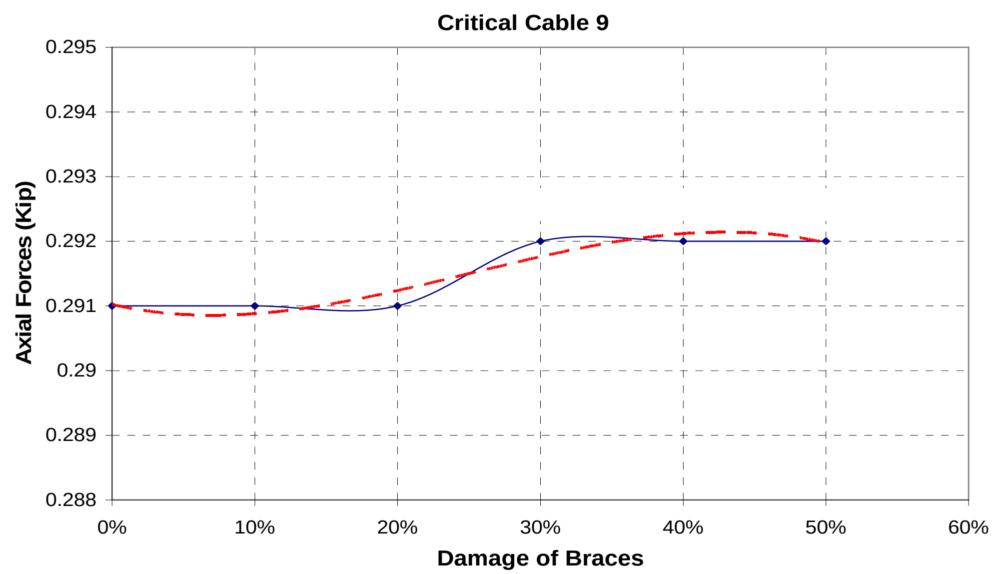


Figure 6.14. Variation of Axial Force in Cable 9 with Damage to Braces

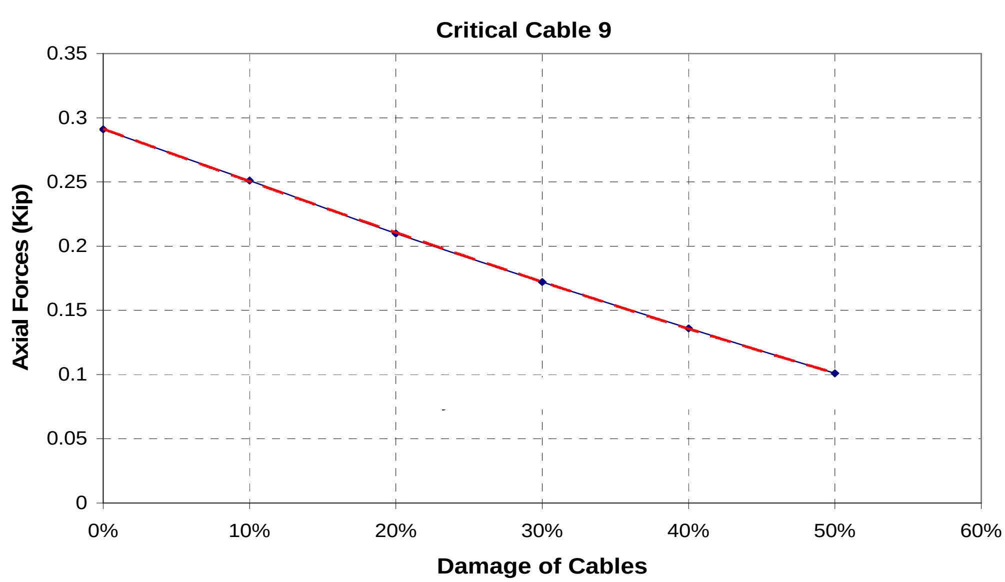


Figure 6.15. Variation of Axial Force in Cable 9 with Damage to Guy Cables

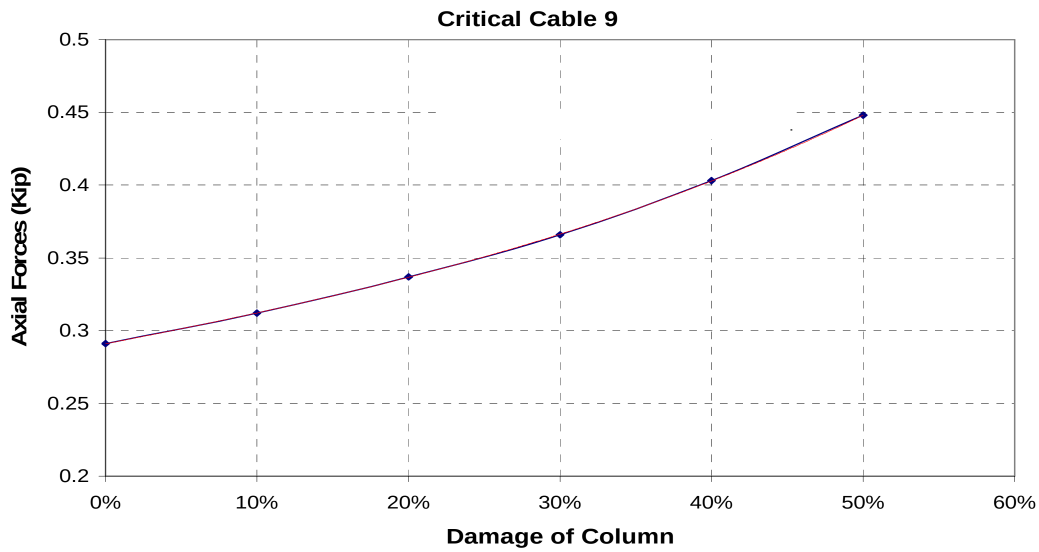


Figure 6.16. Variation of Axial Force in Cable 9 with Damage to Legs

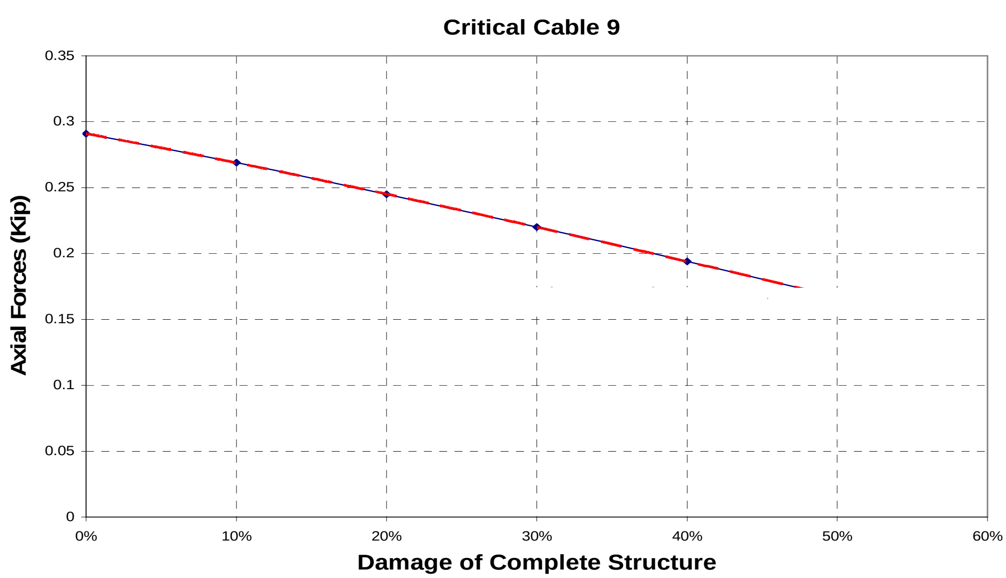


Figure 6.17. Variation of Axial Force in Cable 9 with damage to all members

6.4.3 Effect of Component Damage on Tower Legs

The effect of damage on the forces in the tower legs was also investigated. The critical legs were defined by evaluating the axial forces in the tubes for the undamaged tower under seismic loading (Figure 6.18). The lower legs were stressed the most and thus were selected for further evaluation. The axial force in the bottom legs with time is shown in Figure 6.19. Variation of the axial force in the legs with the variation in damage to individual components of the tower as well as to all members of the tower is shown in Figures 6.20 through 6.23.

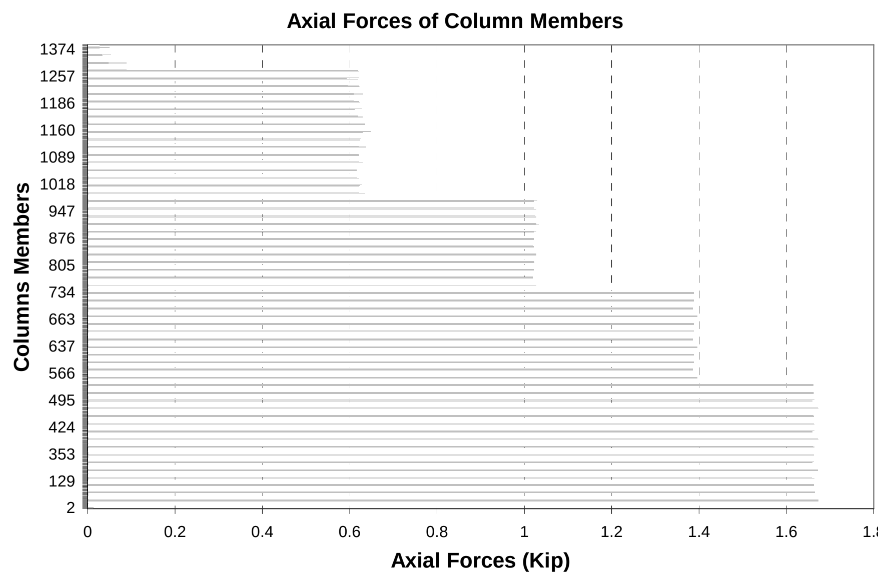


Figure 6.18. Axial Forces of All Leg (column) Members

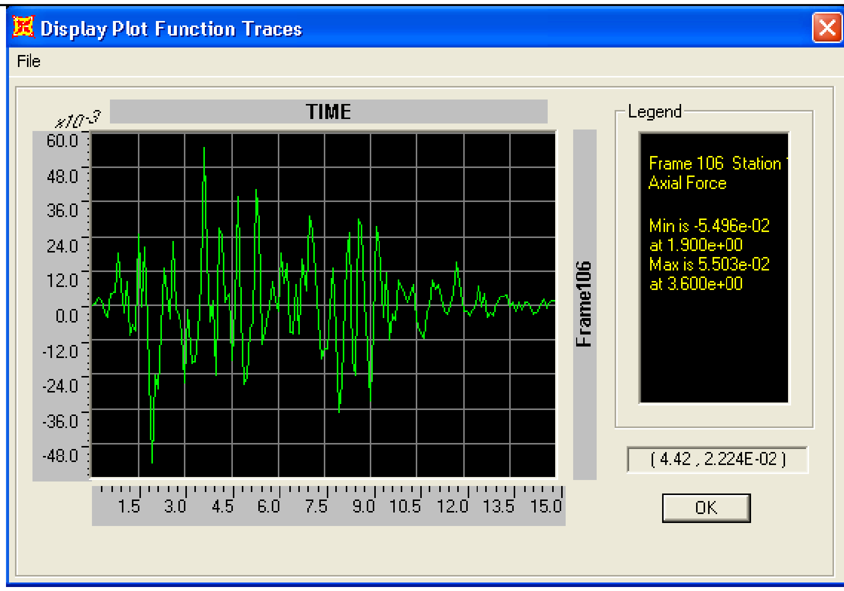


Figure 6.19. Axial Forces with Time for Leg Member # 106 for Time History Function

Changes in axial forces are much more pronounced in the legs when damage is introduced to tower components than in any other component studied. Damage to the braces causes minimal decrease in the axial forces of critical leg member # 106, as seen in Figure 6.20. When the cables are damaged, more of the load must be taken by the legs of the tower. Therefore Figure 6.21 shows the increase of approximately 0.5 kip in axial force in critical leg # 106. When the leg columns are damaged the axial forces in leg # 106 decreases by 0.4 kip. When the legs are damaged other components of the tower must compensate for the lack of strength in the legs. Therefore the other components picked up more load making the axial forces in the critical leg decrease, as seen in Figure 6.22. When all components of the tower are damaged, the axial forces can be seen to increase approximately 0.25 kip in Figure 6.23.

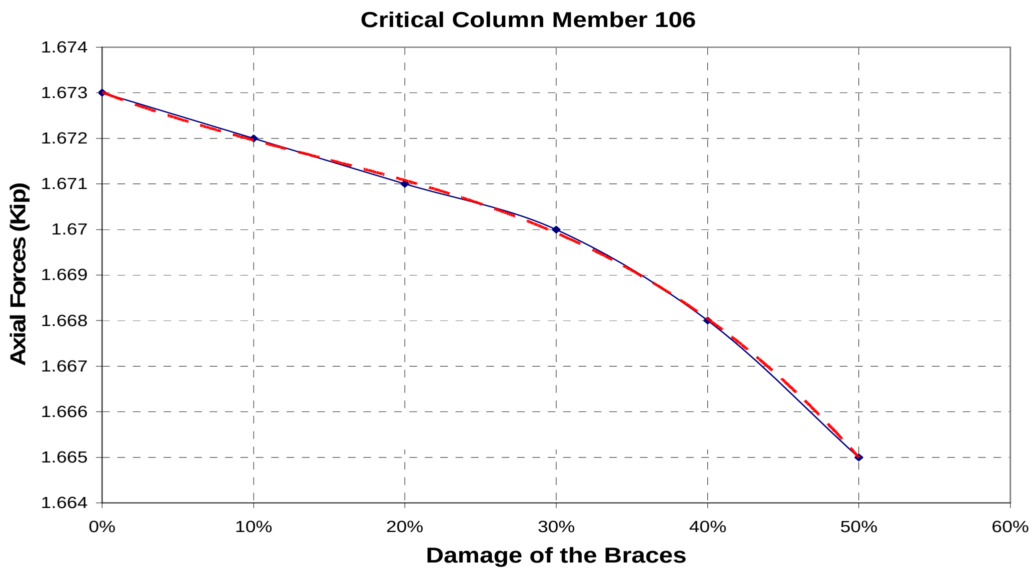


Figure 6.20. Axial Force in Leg Member # 106 with Damage to Braces

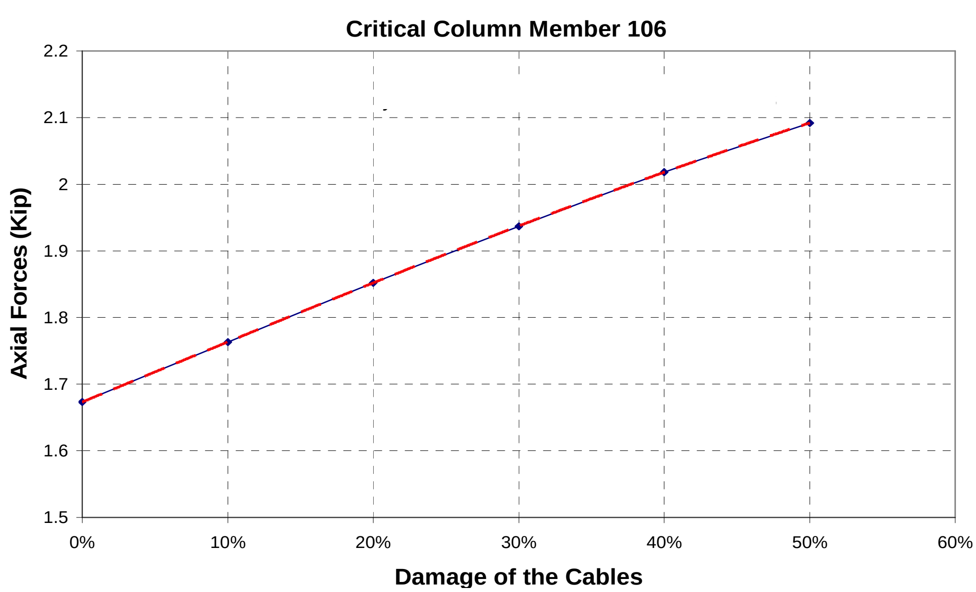


Figure 6.21. Axial Force in Leg Member #106 with Damage of Cables

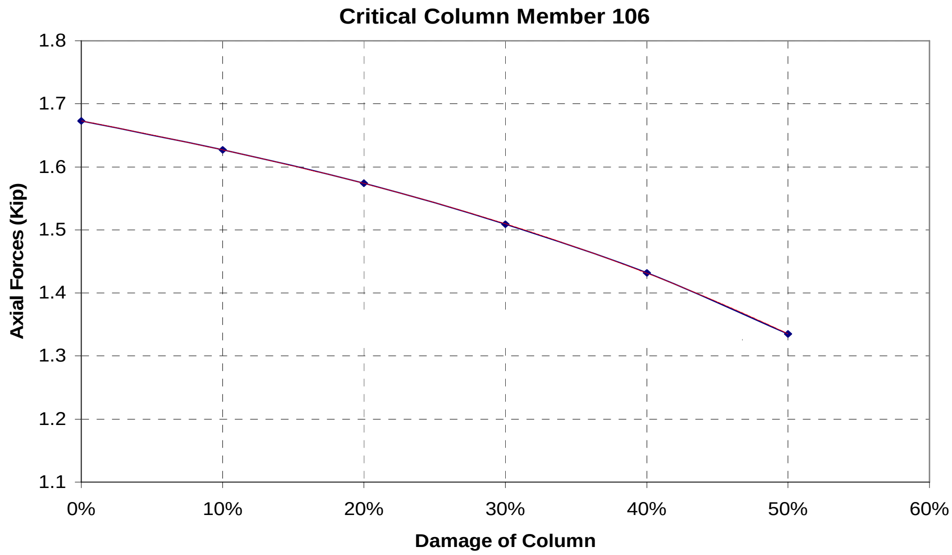


Figure 6.22. Axial Forces in Leg Member # 106 with Damage of Column

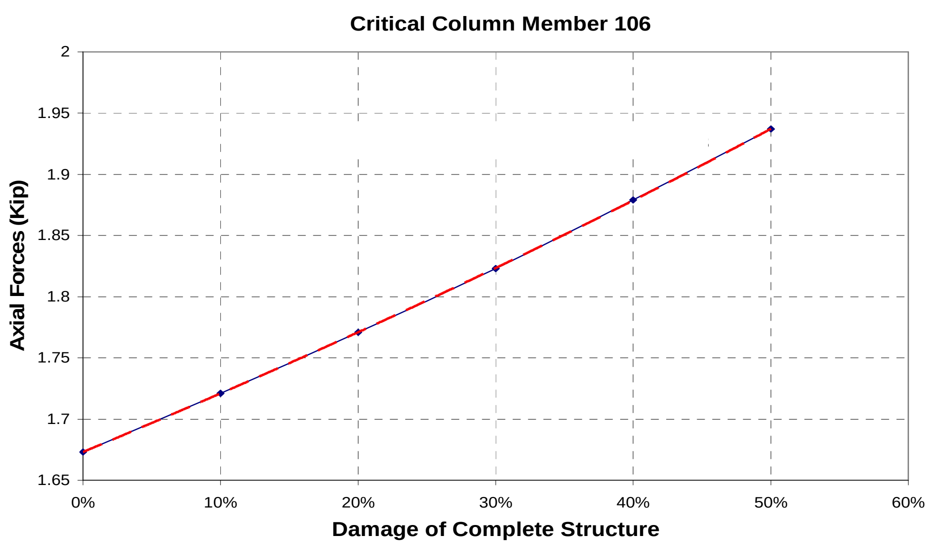


Figure 6.23. Variation of Axial Force in Leg #106 with damage to all members

6.5 Summary

Deterioration of tower members can be very detrimental to the stability of the structure. A parametric study of the Taum Sauk tower was done and analyzed under wind, ice and seismic load cases determined from the TIA-222 F code. The members cross sectional areas were lowered by increments of 10%. Therefore each member was damaged 10%, 20%, 30%, 40% and 50%. Tests were run using ERITower for wind and ice analysis and SAP for the seismic analysis. Tests were performed by introducing damage only on one type of member and leaving the other members at 0% damage and also damaging all members equally at the same time.

For the parametric study done using ERITower to view the effects of wind and ice loading, it was found that the diagonal braces are the most stressed members in the tower structure. Due to this fact, the bracing causes the tower to fail at very little damage, 0-9%. The guy cables also seemed to be a controlling factor in the stability of the tower. The tower failed at around 25% of cross sectional reduction of the cables. The damage to the cables actually relieved loads on the bracings and legs, but still failed at a low amount of damage. The other member types in the tower reacted well to deterioration. The legs of the tower can take up to 42% deterioration in cross sectional area before failure occurs.

The parametric study done using SAP viewed the effects of cross sectional deterioration under seismic loadings. It is seen that there is an approximate 0.5 kip increase in axial force in the legs when the guy cables are damaged, and a 0.25 kip increase in the legs when the whole tower is damaged to 50% original cross sections. A 0.15 kip increase in axial force in the guys is seen when the legs are damaged to 50% their original cross sections. Other increases are seen in the components due to damage, however none as significant as the before mentioned. Therefore it can be said that the legs and the guys control the towers structural integrity for this parametric study. Damage to either the tower legs or the guys can cause significant increases in axial forces when subjected to seismic loadings.

From the results it can be seen that the deterioration of the members can be very detrimental to the structural integrity of the tower. Specifically the diagonal bracing for wind and ice loads, while guys and legs control tower stability during seismic loads. Knowing that the tower system was built in the 1950’s, it is imperative to identify existing tower conditions through field inspections, destructive and non destructive evaluations, and condition indexing.

7 Conclusions and Recommendations

7.1 Conclusions

The Missouri Department of Transportation radio communication tower network was developed in the 1950 and 1960s as a civil defense system. The network is now used by state emergency crews and law enforcement agencies on a daily basis. The towers were developed using the TIA-222-C (1976) version of the tower building code which is now out of date. The current code used in practice today is the TIA-222-F (1996). The main problem for the Missouri Department of Transportation is rating the towers with respect to importance to the network and condition of the tower, so that steps can be taken to maintain or renovate existing towers. This thesis developed a systematic evaluation and assessment method to provide information necessary for repair and maintenance of the network and also help develop a condition indexing tool created by Tulasi (2005).

This thesis encompassed the modeling of two towers in the tower network, one in Kansas City and one on Taum Sauk Mountain. The Kansas City tower is a 250 foot, 3-sided, free-standing tower, while the Taum Sauk tower is a 150 foot guyed tower. The Kansas City tower is considered to be in good shape, while the Taum Sauk tower is considered to be vulnerable to problems due to its close proximity to the New Madrid fault line. Therefore the Kansas City tower was analyzed only for wind and ice using the TIA-222-F version of the code. The Taum Sauk tower was analyzed for wind and ice loadings, and also for seismic loading. Along with seismic analysis, a parametric study of the Taum Sauk tower was also completed.

Both towers were modeled in both ERITower and SAP2000. ERITower is a tower analysis tool that utilizes the TIA-222 tower code. This tool can perform wind and ice loadings on the tower; however it does not have the capability to perform dynamic analysis. Seismic analysis was performed by SAP2000 in accordance with the TIA-222-F. To make sure that ERITower and SAP provided accurate results, a simple tower was created and modeled in both modeling tools. The two models were loaded identically and found to produce identical results which can be found in Chapter 3.

The Kansas City tower was studied to determine if the member sizes used for the tower would withstand the loadings created using the TIA-222-F, along with the added user loads such as antennas. This tower was not loaded with seismic loads, since it is not in an active seismic location. The tower was found to pass and be at an 88.9% tower capacity. The controlling members of the tower were found to be the diagonal bracing between the legs of the tower. The other members of the tower were found to be at most 51.1% of their capacities.

The Taum Sauk tower was analyzed using the TIA-222-F code for wind, ice and seismic loadings. ERITower was used to complete the wind and ice loadings, while SAP2000 was used to compute the seismic loadings. The parametric study was completed using both tools. To create the parametric study, the components of the tower, legs, braces and guys, were decreased in cross section from 10% to 50% by 10% increments. The tower was then loaded by introducing damage to only one type of component at a time for damage from 10% to 50%. Finally the tower was loaded with all of the components being damaged at the same time at the same percentage of lost cross section. These results were then used to determine how a tower will react with specific damage to the legs, braces, guys or combinations of all three. This data was then used to help create a conditioning index for the tower network (Tulasi, 2005).

The parametric study performed with ERITower was used to determine which component controlled the tower when wind and ice loads were applied. When damage was introduced to all tower members, the failure of the diagonal brace always controlled the tower rating. For example, at 10% damage to the tower, the tower failed because the diagonal brace at an elevation of 100 to 110 feet exceeded its capacity by over 5 times. Results shown in Chapter 6, Table 6.4a exhibit somewhat irregular behavior, which is likely a result of significant overstressing after 10% damage to the bracing is reached. Results in Chapter 6, Table 6.4b, which consider bracing damage on a more realistic scale ranging from 0% to 9%, illustrate that the tower capacity appears to approach 100% for bracing damage less than 10%.

Additional observations include the following:

- The diagonal braces are the most stressed members in the tower structure at 70% of their capacities.
- Damage to the guys only causes the tower to fail at about 25% reduction in the guy cross section and corresponding tension. Damage to the guys relieves the loads on the bracings and legs. Since this is a guyed tower, damage to the guys causes the tower rating to exceed 100% capacity and collapse at about 25% damage to the guys.

- Little damage to the braces (< 10%) causes the tower rating to exceed its capacity. This indicates that the braces can significantly result in tower instability with very little damage.
- Damage to the tower legs as high as 42% will result in the overall tower to fail. Thus, the damage to the legs is not as critical as that to braces and cables. It is important to note though that tower legs can corrode faster if moisture gets trapped inside the tower legs.

The parametric study performed with SAP2000 was used to determine which components controlled the tower when seismic loads were applied. Unlike ERITower, SAP does not state whether or not the tower has failed. Instead, axial forces in each component are output. From these outputs, the critical components can be determined by seeing where the greatest increase in axial force is seen when damage is introduced to specific components from 0% to 50%. When damage was introduced in the guys, the critical leg had approximately a 0.5 kip increase in axial force. Also the critical leg had around a 0.25 kip increase in axial force when the entire tower was damaged. The other significant increase in axial force was a 0.15 kip increase in the guys when the legs were damaged. Other increases in axial force due to damaging the tower components were minimal when compared to the previous increases. Therefore the legs and guys are the controlling components of the Taum Sauk tower, when seismic loads are applied.

The studies on the Kansas City and Taum Sauk towers have shown that all the components of the tower are important to the structural integrity of the tower. However when prioritizing towers it is necessary to know what components affect the structure the most if damage occurs. It has been shown that for a tower like the Kansas City tower, the diagonal bracing tends to control the ability of the tower to withstand wind and ice loadings. For a guyed tower, the diagonal bracings again are the controlling components when wind and ice loadings are applied. However, if seismic loads are also a factor, the condition of the guys and legs should also be considered important to the structural integrity of the tower.

7.2 Recommendations

To create more accurate results for the condition indexing system, a few topics need to be addressed further. Following are several recommendations for the future that could make the results more complete.

- SAP2000 was limited in that it was unable to create cable members. Therefore the guys were modeled as beams and had compressive axial forces present. The compressive axial forces were neglected from the results. A better way of modeling the Taum Sauk tower could be found, possibly using a different modeling tool, that would give more accurate results when the tower is subjected to seismic loadings.
- From a field analysis of the Kansas City and Taum Sauk towers, it was determined that the member sizes used to build the towers were not always the same as the member sizes on the original drawings of the tower. Therefore a field analysis of the member sizes and deterioration level should be completed. New models can then be created that will give a more accurate representation of the towers respond to different load cases.

- Deterioration was modeled by reducing the cross sectional area of the entire member. Deterioration will actually be more prominent close to the connections, and will not be uniform throughout the members. A more exact way to model damage could be created by determining where deterioration is most prominent. This will allow the surface areas of the members to be modeled more correctly which will create more accurate loadings on the tower.
- Destructive and/or non-destructive evaluations should be performed to determine the actual condition of all components of each tower. Members of each a tower could be taken out and replaced so tests could be run to find the actual strength of a member subjected to corrosion. Such information can be used to modify the analytical models and arrive at a better assessment of the towers.

Appendix A. Structural Drawings for Taum Sauk Tower

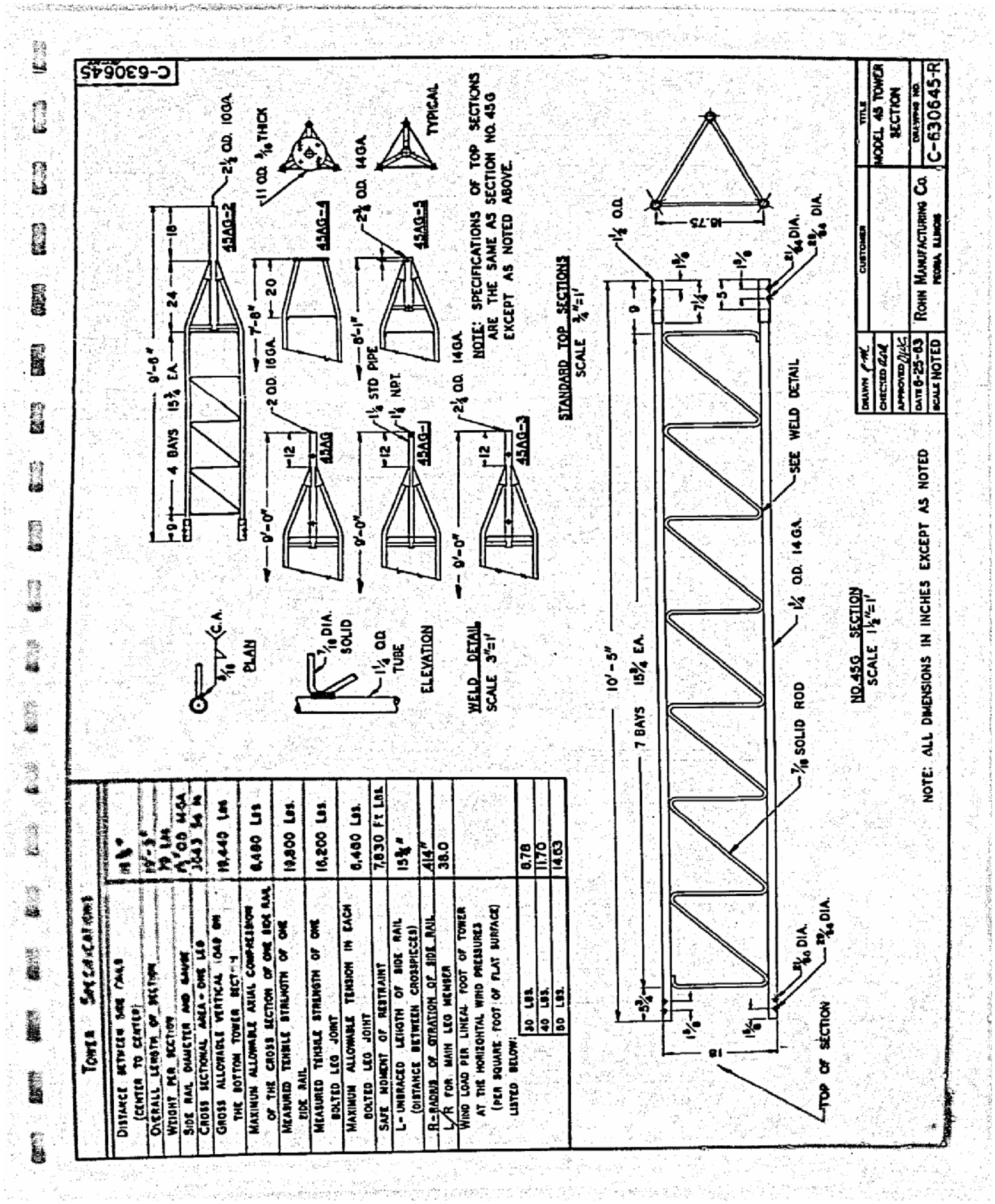


Figure A1. Taum Sauk Structural Drawings

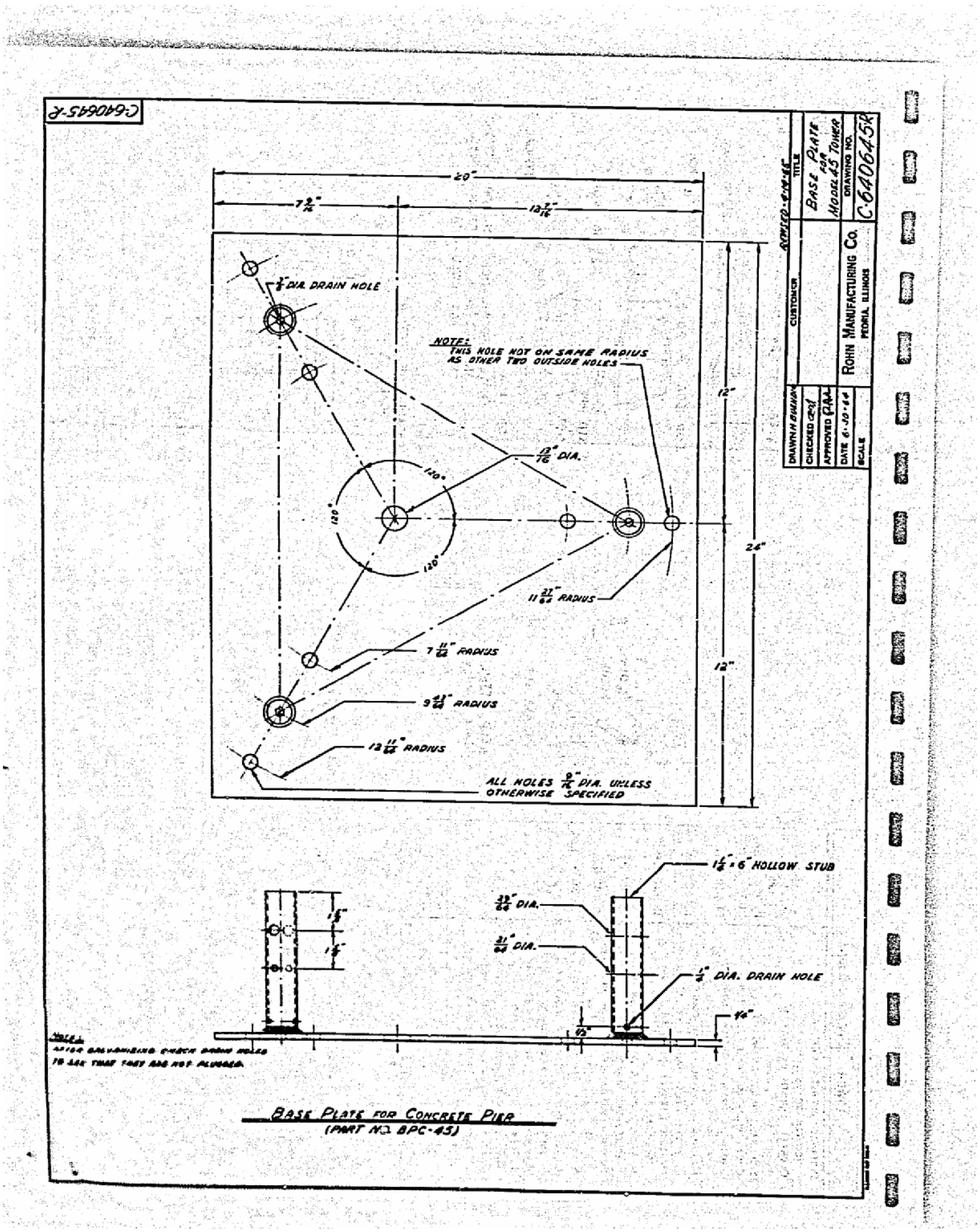


Figure A2. Taum Sauk Structural Drawings

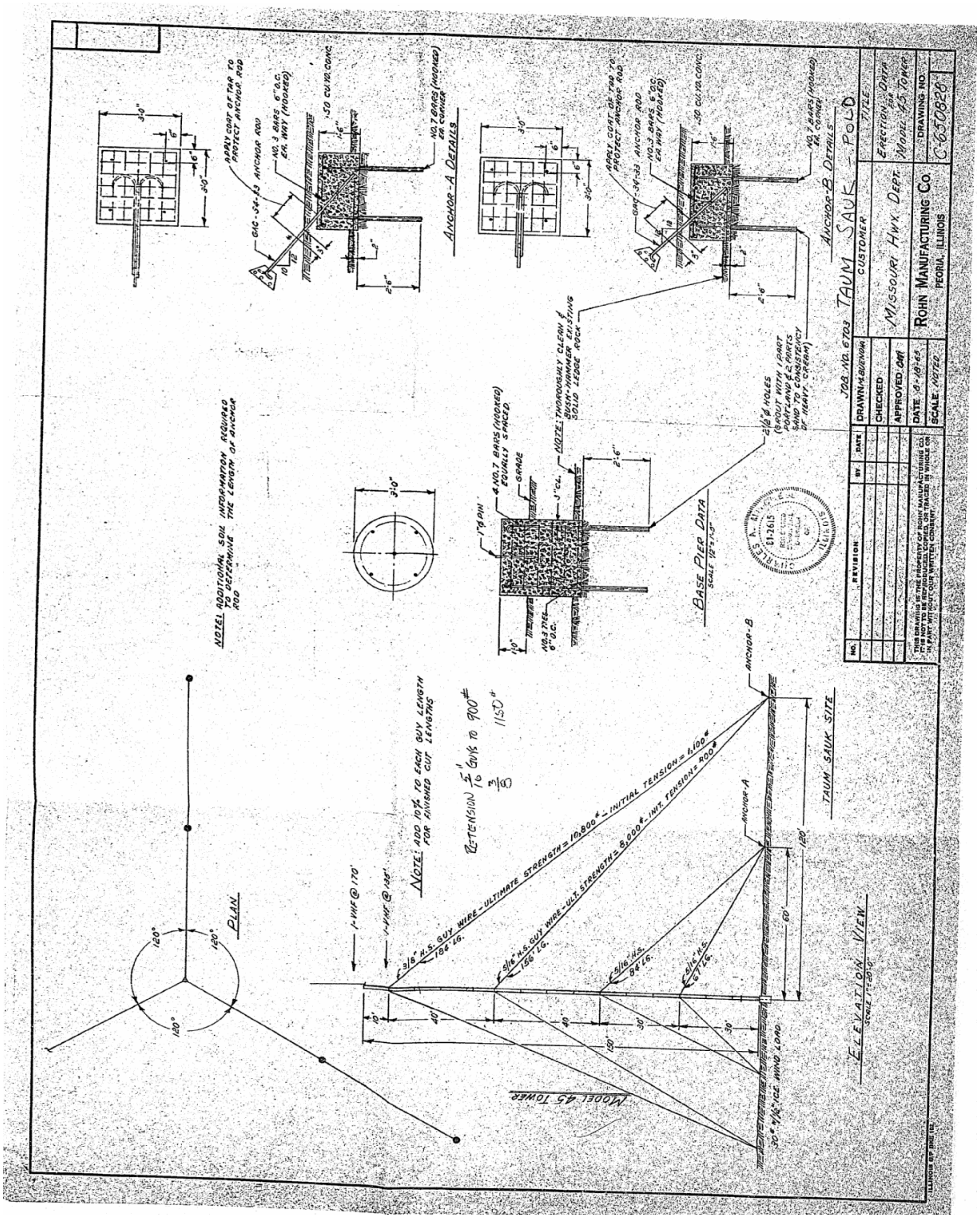


Figure A3. Taum Sauk Structural Drawings

Appendix B. Modes of Vibration for Taum Sauk Tower

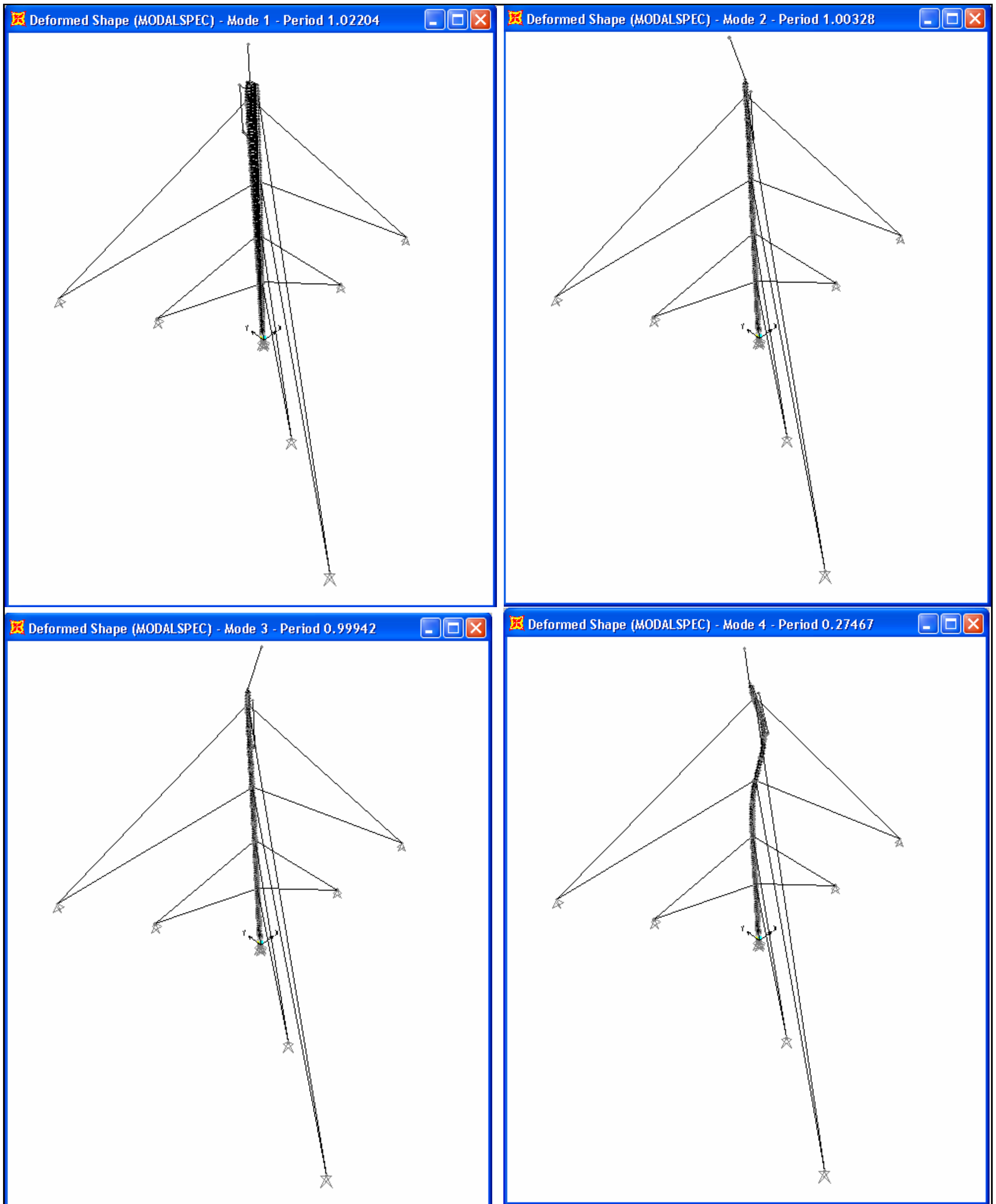


Figure B1. Mode shapes 1 - 4

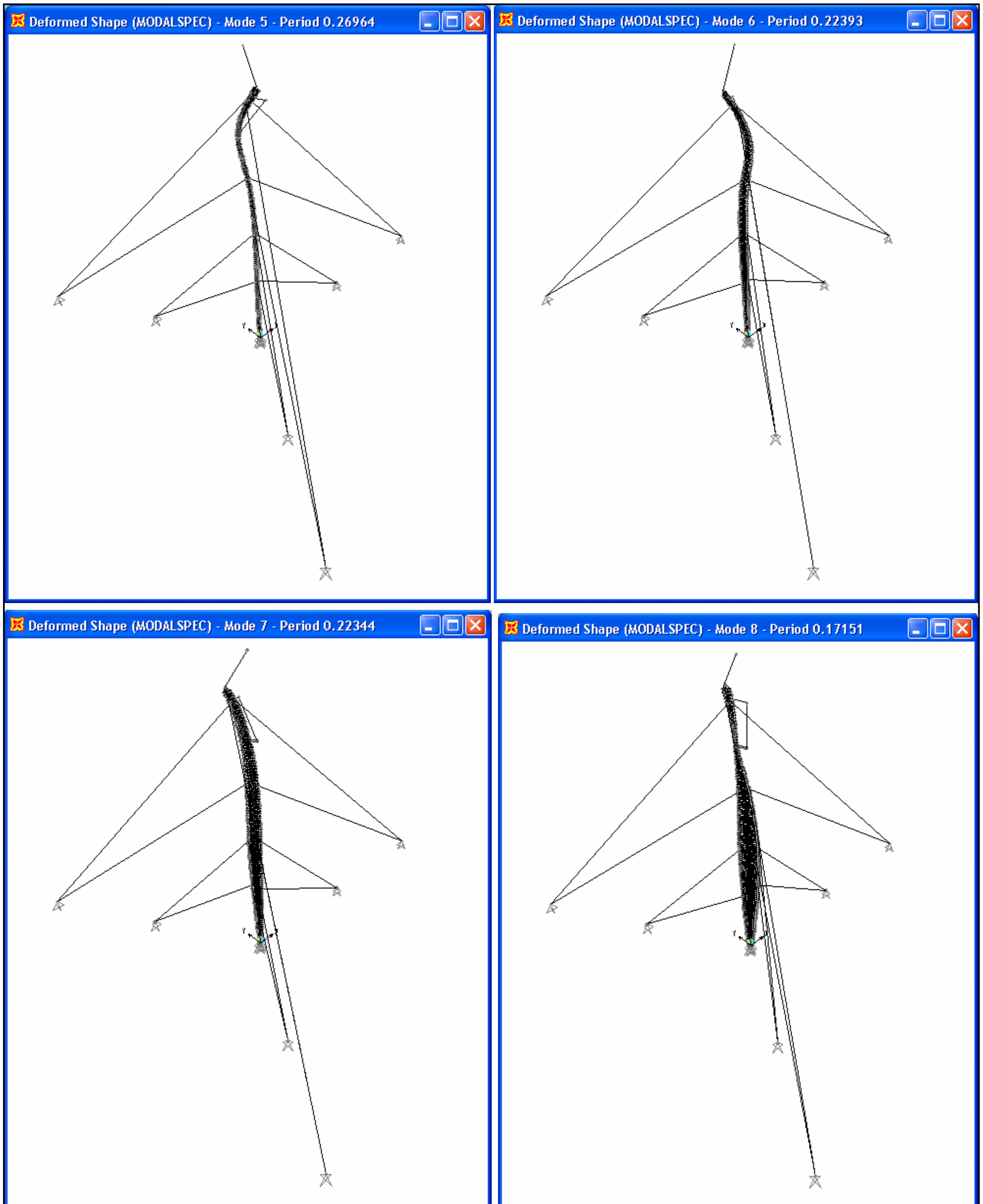


Figure B2. Mode shapes 5 - 8

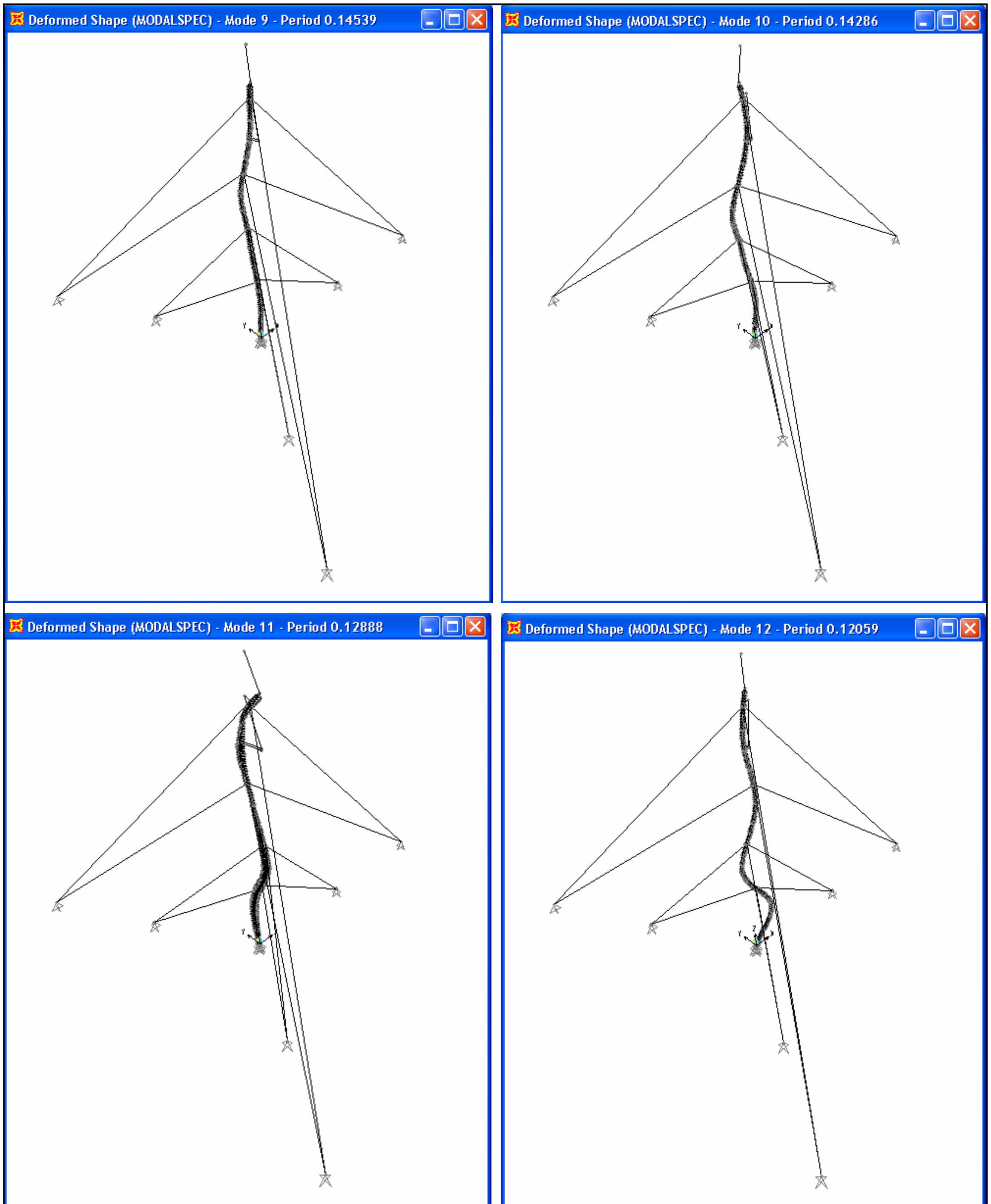


Figure B3. Mode shapes 9 - 12

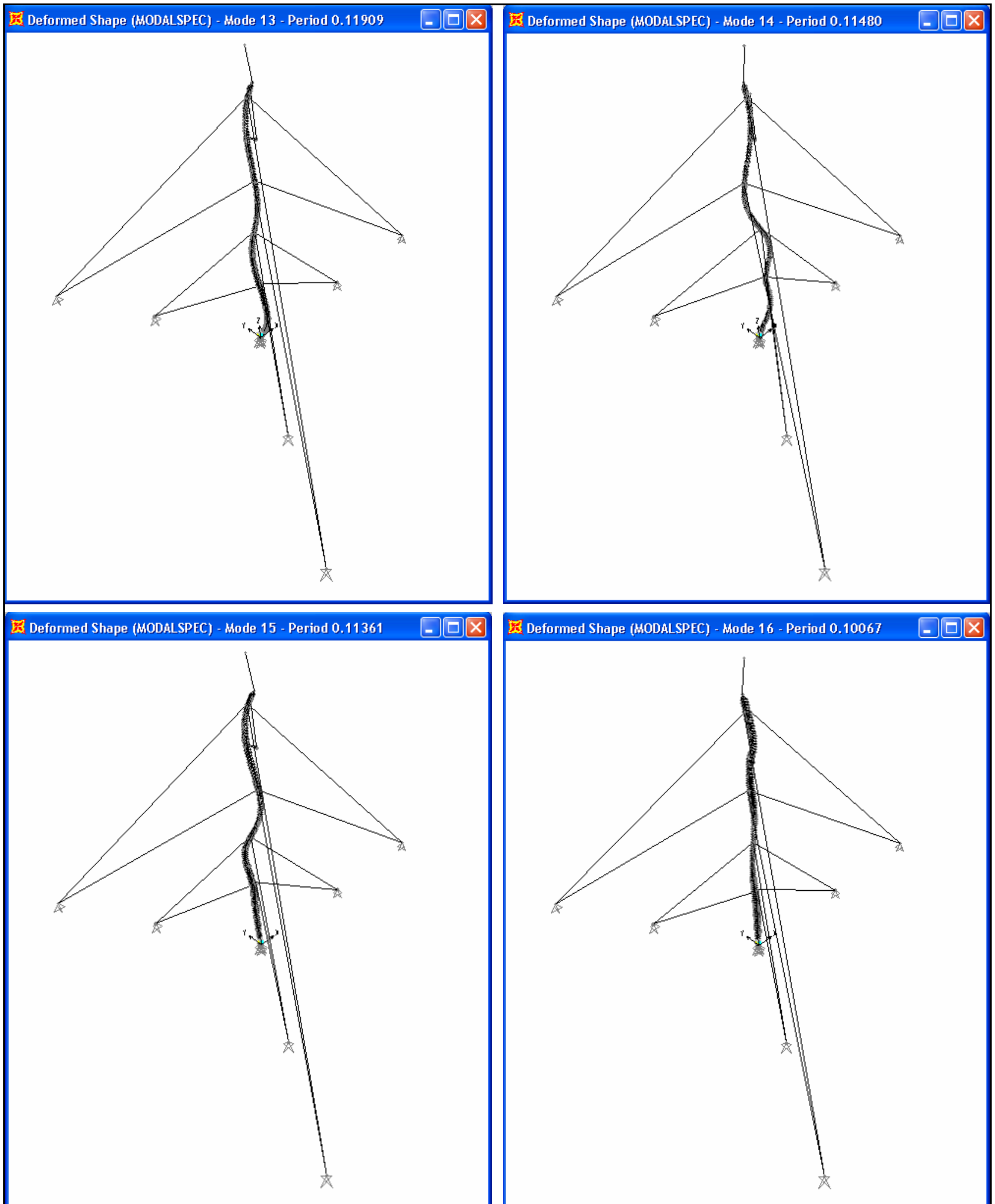
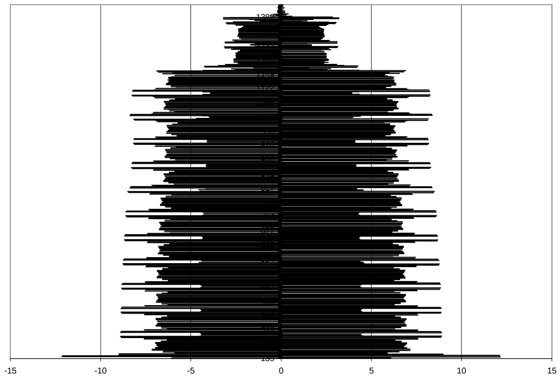


Figure B4. Mode shapes 13 - 16

Figure B5: Axial Forces for Braces for All 16 Modes for Response Spectrum Function



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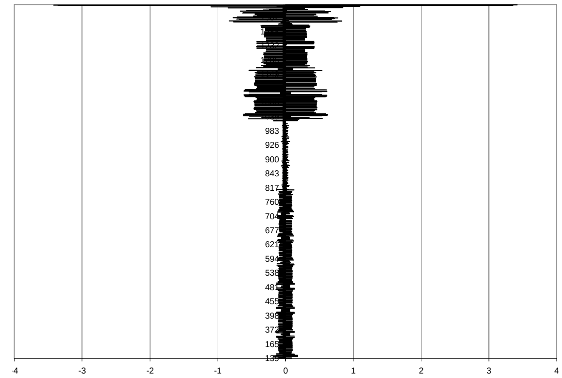

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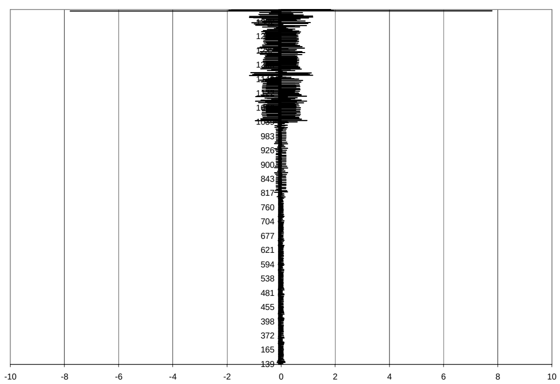

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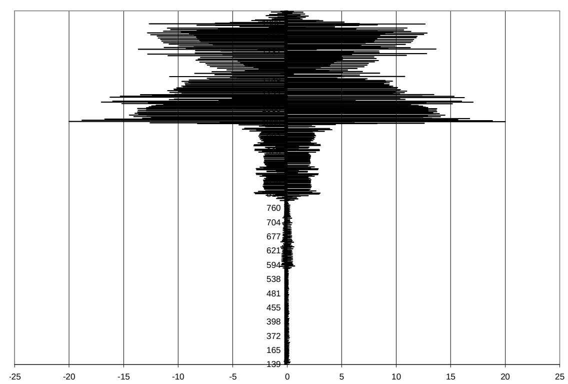

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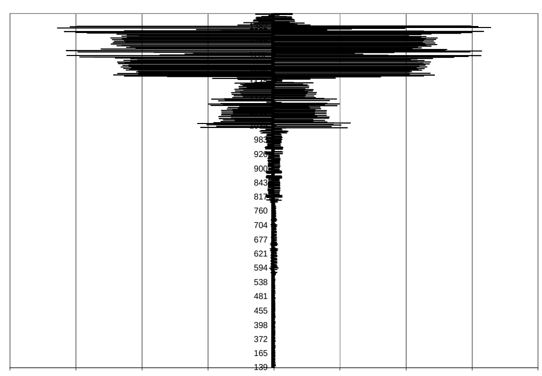

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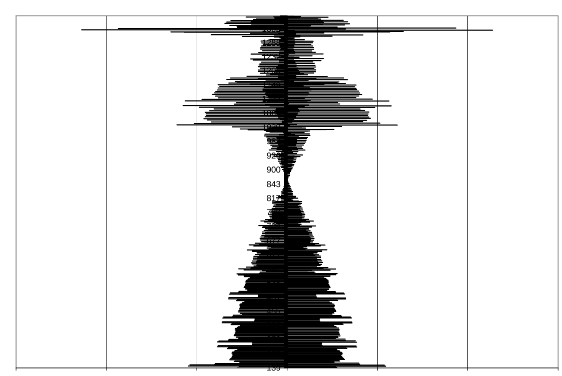

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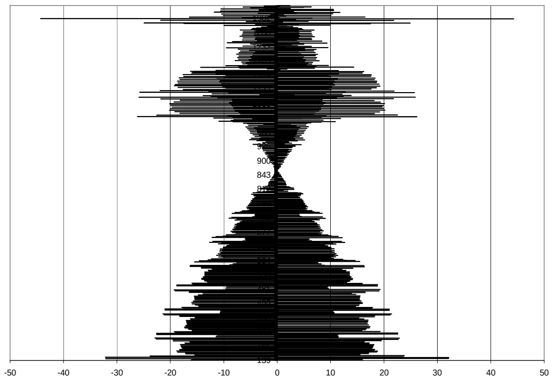

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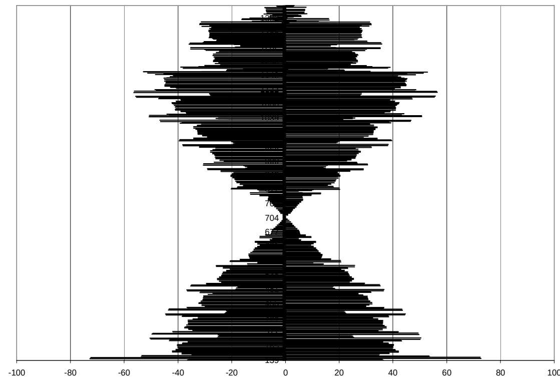

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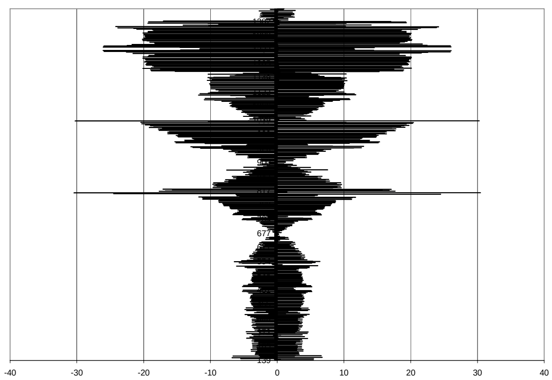

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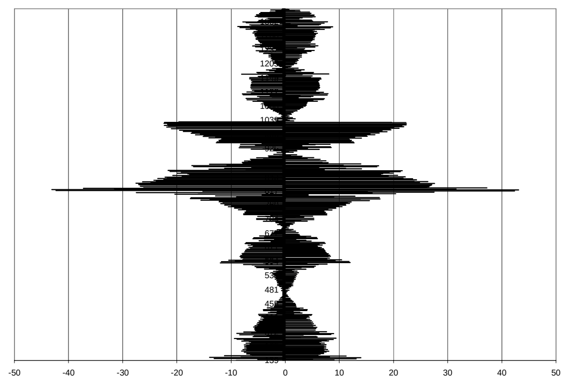

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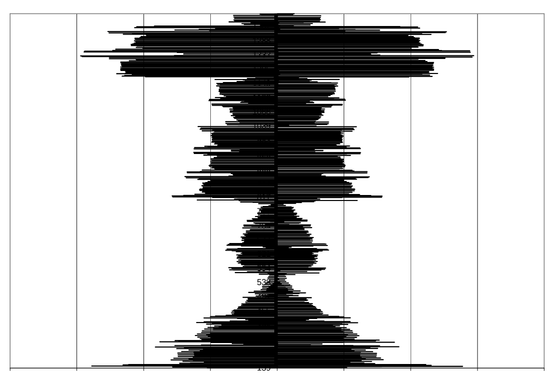

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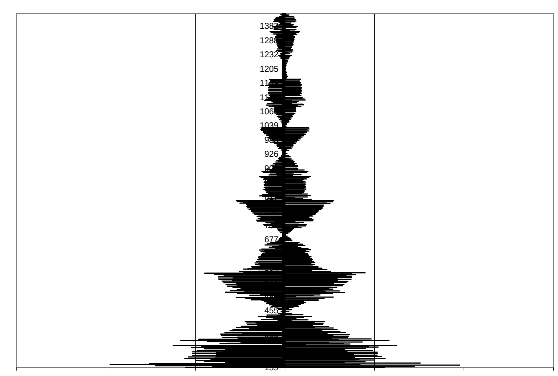

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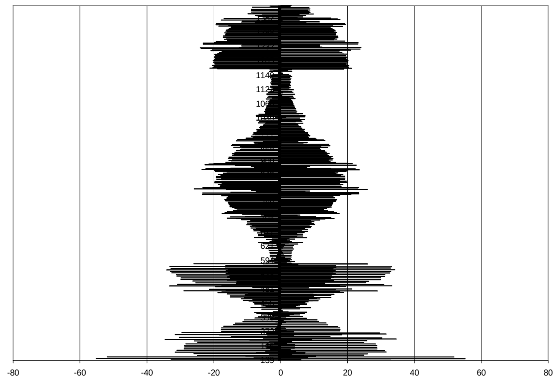

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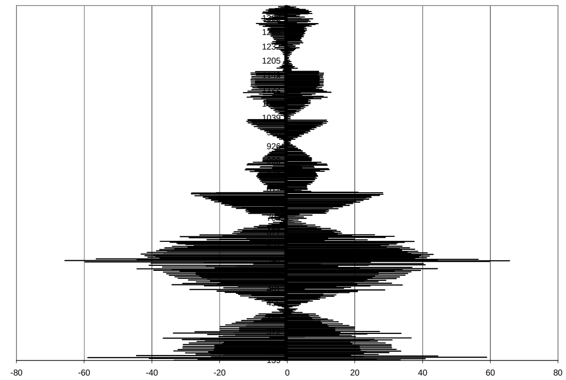

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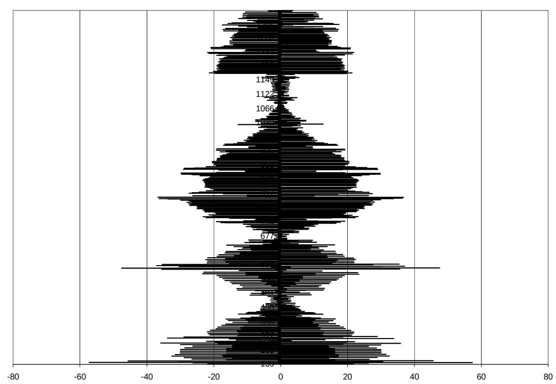

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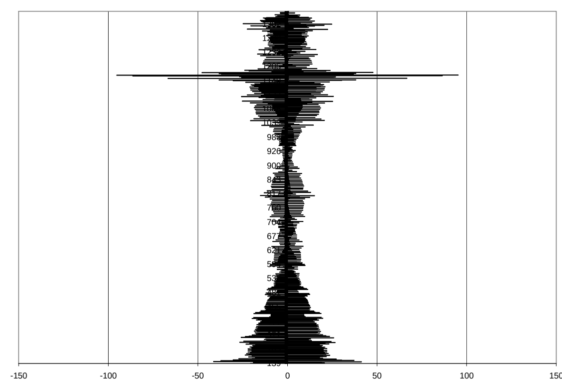

Mode 16

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