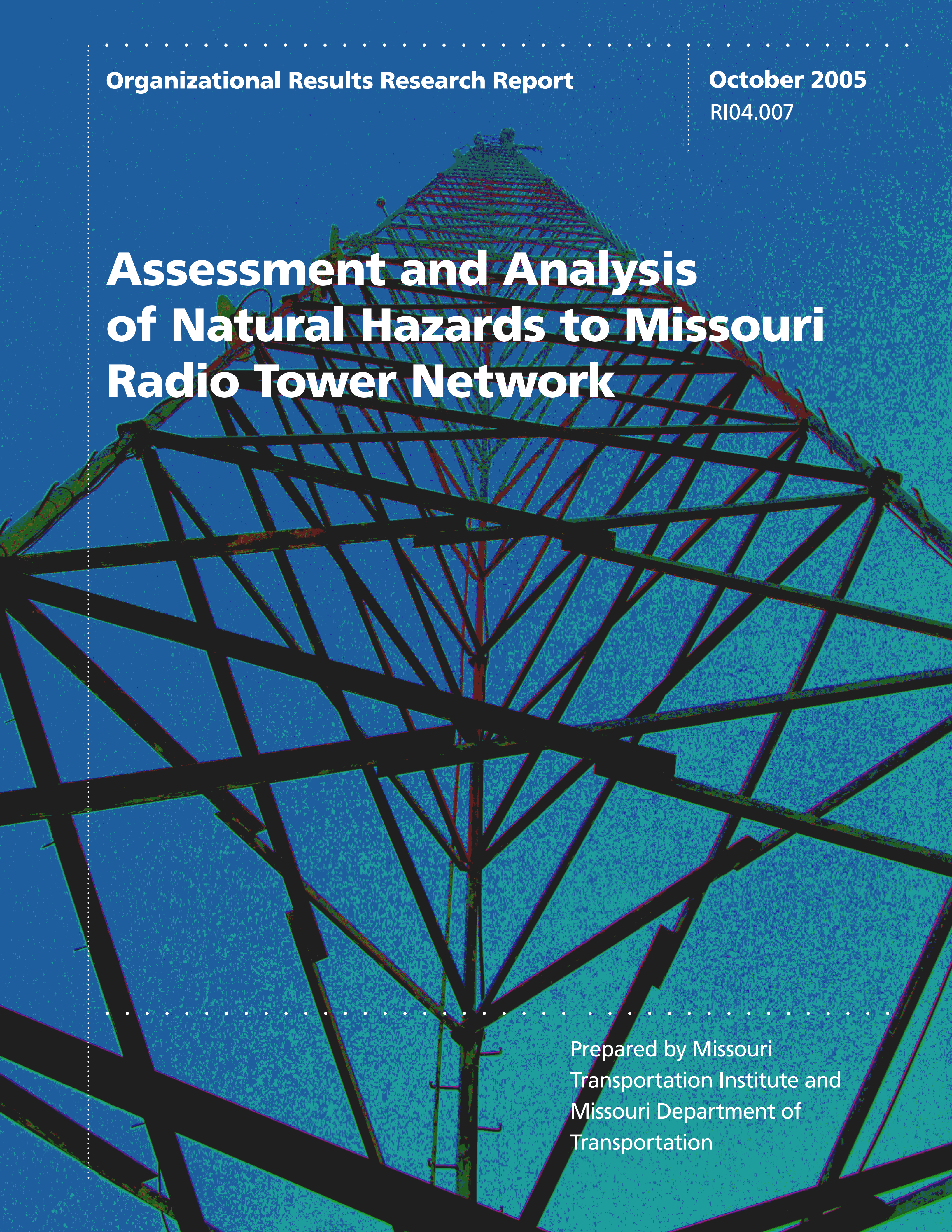

Organizational Results Research Report

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Assessment and Analysis of Natural Hazards to Missouri Radio Tower Network

Prepared by Missouri Transportation Institute and Missouri Department of Transportation

Final Report

Research Investigation R104-007

Condition Assessment and Natural Hazards Analyses for Communications Towers

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16. Abstract. Design and construction of the MoDOT two-way radio network was initiated in the 1950's and 60's and was motivated primarily by the need to provide a state-wide communication system for civil defense related issues. Today, the system is one of the largest networks in the state for providing voice and data communications associated with daily field operations as well as during times of emergency. Increasing reliance on the network is envisioned in the future to support wireless interoperability (e.g., with police and fire) and to support an increasing load of data communications for intelligent transportations system (ITS) infrastructure.

Growing concern regarding the network's performance during natural hazard events, most notably the possibility for significant earthquakes originating from the New Madrid Seismological Zone, has stimulated the desire for a comprehensive analysis of the network and the development of systematic asset management tools. A recent statewide emergency preparedness exercise involving a mock 6.7 magnitude earthquake revealed that the chief obstacle encountered as field crews attempted to communicate with each other was failure of the radio towers. Many of the towers are over 40 years old and in relatively poor physical condition.

The primary objectives of this project were (1) to develop a rational condition indexing (CI) system as an asset management tool that may be used to systematically quantify the physical condition of towers in the network; (2) to conduct detailed dynamic and static structural analyses of key towers under seismic, wind and ice loading; (3) to evaluate the general effects of physical deterioration (e.g., corrosion related to aging) on tower dynamic response and stability; and (4) to develop a centralized electronic database.

A CI system has been developed to quantify the physical condition of guyed communications towers. Use of the proposed system is demonstrated for two towers (Taum Sauk and Ashland) selected to represent towers in relatively poor and relatively good condition, respectively. Results from structural analysis indicate that the Taum Sauk tower (as built condition) is not loaded to near its capacity under simulated seismic loading. The Taum Sauk tower is shown to pass TIA-222-F code specifications with respect to wind and ice loading. Deterioration to the Taum Sauk tower was simulated by reducing the cross sectional areas of the guy cables, diagonal braces, and axial leg members. A free standing tower in Kansas City was also analyzed and is shown to be loaded to 2% over capacity according to TIA-222-C but meets the more recent 222-F code requirements. An electronic web-based database was developed for implementation into management of the tower network.

General recommendations for implementing the results of these efforts are provided, which includes a recommendation to consider the proposed CI approach for a wide range of infrastructure managed by MODOT.

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Executive Summary

Design and construction of the Missouri Department of Transportation (MoDOT) two-way radio network was initiated in the 1950's and 1960's and was motivated primarily by the need to provide a state-wide communication system for civil defense related issues. Today, the system is one of the largest networks in the state for providing voice and data communications associated with daily field operations as well as during times of local, regional, and national emergencies. Increasing reliance on the network is envisioned in the future for additional support of wireless interoperability among state agencies outside MoDOT during emergencies (e.g., fire, police) and to support a rapidly increasing volume of data communications associated with advanced field monitoring systems and intelligent transportations system (ITS) devices.

The current and anticipated reliance on the communication system has generated significant motivation for a comprehensive physical analysis of the aging network and the development of systematic tools for managing the infrastructure that supports it. Most notably, this includes approximately 50 communications towers located throughout the state, many of which are over 40 years old and in relatively poor physical condition.

The towers represent a major asset for MoDOT. Assessing their capability to withstand extreme events is currently a high priority asset management requirement. Concern regarding tower performance during natural hazard events, most notably the possibility for significant earthquakes originating from the New Madrid Seismological Zone, but also during natural environmental loading such as wind and icing events, has stimulated the desire for systematically assessing the current physical conditions of the towers, including detailed analyses of tower response under dynamic and static environmental loads.

The primary motivation for this project was to respond to concerns and uncertainties regarding the current physical condition of the tower network and the associated performance of key towers during environmental loading events (seismic, wind, and ice). Objectives of the project were:

1) To develop a rational, quantitative condition indexing (CI) system that may be used as an asset management tool to systematically quantify the current physical condition of towers in the network;

2) To conduct detailed dynamic and static structural analyses of key towers under seismic, wind and ice loading;

3) To evaluate the general effects of deterioration on tower dynamic response and stability;

4) To develop a centralized electronic database for more effectively managing the tower network.

A CI system has been developed to quantify the physical condition of guyed communications towers. The proposed system is a simplified version of a more complex asset management tool developed by the US Army Corps of Engineers. Use of the CI system is demonstrated for two

towers in the MoDOT network. These include towers at the Taum Sauk and Ashland sites, which were selected to represent towers in relatively “poor” and relatively “good” condition, respectively. The overall CI for the Taum Sauk tower is 54 out of 100, which corresponds to “Fair: moderate deterioration but function is still adequate.” The overall CI for the Ashland tower is 85 out of 100, which corresponds to “Excellent: no noticeable defects; some aging or wear may be visible.” Subsequent weighting factors are suggested for accounting for the relative importance of each tower to the health of the overall communication network. Classification of the Taum Sauk and Ashland towers may be treated as example cases to facilitate implementation of the proposed asset management tool to other towers in the state.

Results from dynamic structural analysis indicate that the Taum Sauk tower in its as-built condition is not loaded to its capacity under simulated seismic loading according to Uniform Building Code procedures. The Taum Sauk tower also passes with respect to the design wind and ice loading events according to TIA-222-F code specifications. Subsequent deterioration to the tower was simulated by reducing the cross sectional areas of the guy cables, diagonal braces, and axial leg members. Qualitative observation of the tower in its current condition indicates that the existing level of deterioration in these members does not exceed the critical deterioration levels identified in the analysis, thus suggesting that the tower is not likely to fail during anticipated environmental loading events (seismic, wind, and ice). The Kansas City free-standing tower is loaded to 2% over capacity according to TIA-222-C code requirements for wind and ice but meets the more recent 222-F code requirements. Results from the structural analyses provide quantitative guidance to assist in making maintenance decisions for specific components of guyed and free-standing towers and to predict their expected performance during seismic, wind, and ice loading events.

An electronic web-based database was developed for more efficiently managing the tower network. The database includes an interactive map of Missouri where the user may click on a select tower location to access information related to the physical specifications of the select tower (e.g., type, height, etc), scanned copies of the original tower structural drawings, and an interactive screen for entering and updating the current condition index (CI) of the tower. The database may be uploaded to an FTP site for internal distribution.

Key recommendations for implementation include the following:

1) The proposed conditioning indexing system for guyed towers should be expanded and modified for applicability to free-standing and monopole towers. Detailed documentation of the CI development procedures are provided for this purpose.

2) The CI system should be used to rank additional towers in the network. Subsequent inspections and maintenance should be performed on a schedule as follows (from TAI/EIA 222-G):

    a) at a minimum of three-year intervals for guyed masts and five-year intervals for self-supporting structures.
    b) After severe wind, ice, or earthquake loadings

c) Shorter inspection intervals are required for structures in corrosive atmospheres or subject to frequent vandalism.
d) After a change in type, size, or number of appurtenances such as antennas, transmission lines, platforms, ladders, etc.
e) After any structural modifications
f) After any change in serviceability requirements or land use surrounding the structure.

3) The CI for each tower should be updated according to the above schedule and tracked in a centralized location using the electronic database platform provided. Access to the website should be disseminated to regional personnel responsible for tower maintenance.

4) The general approach adopted in developing the proposed CI system (i.e., function-based condition indexing) for radio towers is recommended for consideration of larger-scale implementation to other assets managed internally by MoDOT (e.g., bridges, pavements). The report specifically includes detailed documentation of the CI development procedures in order to facilitate potential larger-scale implementation as an internal asset management strategy.

Specific recommendations for immediate actions to provide greater insurance that the tower network will remain functional following extreme environmental loading events are as follows:

1) Identify 10 to 20 key towers in the network. This task will require detailed knowledge of the network history, scope, and interoperability requirements (e.g., consultation with the primary network operators within MoDOT).

2) Rank the current physical condition of each of the key towers identified in the previous step using the CI system proposed in this document or a similar system developed using the proposed CI framework.

3) Use the rankings identified in the previous step to make executive decisions regarding resource allocation (e.g., establish a cut-off CI value below which REMR actions to mitigate the towers in poor condition will be pursued).

Acknowledgements

This project could not have been completed without the assistance of numerous individuals who contributed their time, efforts, and expertise. Rick Bennett and John Diggs, Missouri Department of Transportation, provided essential technical guidance and facilitated visits to the tower sites. Randy Mayo provided a thorough and insightful review of the final report. Rasid Hadzovic was instrumental in conducting the detailed dynamic and parametric structural analyses presented in Chapter 3. Jeff Jobe, Jinhua Li, Eric Sullins, and Devi Prasad Tulasi conducted many of the detailed aspects of the work. The efforts of Charlie Nemmers, UMC Department of Civil Engineering, and Bill Ankner, Missouri Transportation Institute, are also gratefully acknowledged.

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1. Introduction

1.1 Background and Motivation

Design and construction of the Missouri Department of Transportation (MoDOT) communications (two-way radio) network was initiated in the 1950's and 1960's and was motivated primarily by the need to provide a state-wide communication system for civil defense related issues. The system consists of numerous hand-held and vehicle-mounted radios, radio consoles at each district office and headquarters, base stations located in a number of maintenance buildings and project offices, and approximately 50 free-standing or guyed communications towers located throughout the state. The system provides nearly statewide coverage and is one of the largest two-way communication radio networks in the state. Primary functions of the system currently include the following:

- Voice communications for field operations: Radio communications can be achieved between field vehicles and district offices, maintenance buildings, project offices, etc.
- Data communications for field devices: MoDOT has implemented a number of radio applications for communication with field devices including traffic signals, weather stations, and other traffic devices.
- Communications during disasters and other emergencies: The radio system is designed to provide voice communications during disasters and acts as a backup in the event that telephone/cellular service is not available.

A recent MoDOT Communications System Plan (Bennett and Diggs, 2002) has also defined a long term vision for the communications network to include the above functions as well as increased reliance on the network for:

- Increased support of wireless interoperability between other state agencies and public safety agencies that are part of emergency and disaster response (e.g., fire, police). Currently, very few public agencies can communicate together on radio systems. The MoDOT communications network is a tremendous asset for realizing full interoperability of statewide emergency communications.
- Increased support of wireless data communications for advanced field monitoring devices and intelligent transportations system (ITS) infrastructure (e.g., road/weather information systems, remote traffic sensors, video cameras).

The increasing reliance on the MoDOT communication network envisioned for the future and continuing use in its current capacity has generated significant motivation to assess the physical condition of the infrastructure that supports it. A critical component of the system is a network of approximately 50 communications towers distributed throughout the state (Figure 1). The majority of towers are owned by MoDOT, while others are owned by state agencies such as the Missouri State Highway Patrol (MSHP) and the Missouri Department of Conservation (MDC). The towers support relay stations (repeaters), control and base station, and microwave systems. There is significant variability in tower age, type, height (~ 70 ft to 350 ft), physical condition, underlying soil and rock properties, and significance with regard to successful operation of the overall communication system. Many of the towers are over 40 years old and in relatively poor physical condition.

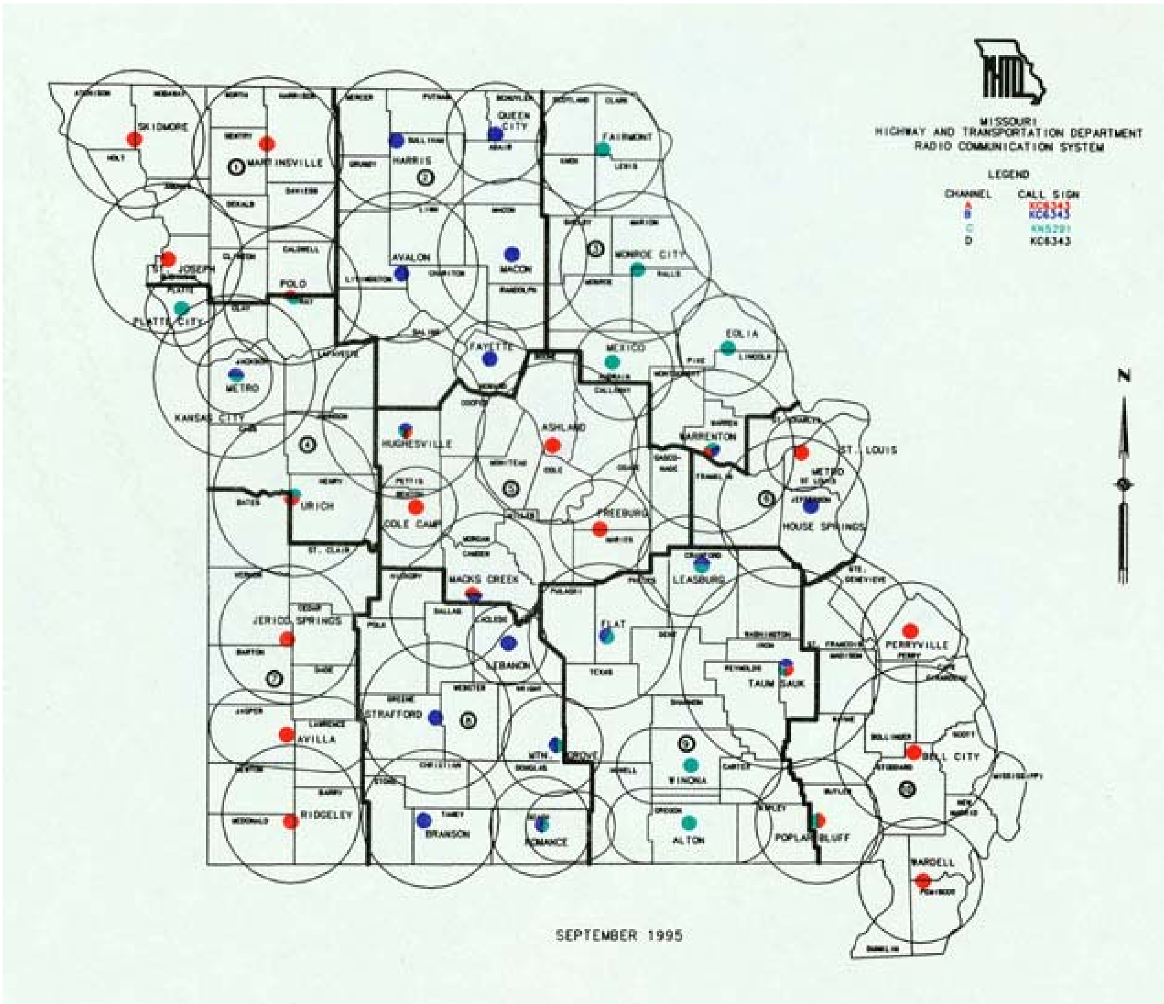


Figure 1.1. Map of MoDOT communication tower locations (Tabulated list in Appendix A).

1.2 Project Objectives and Tasks

Complete replacement or rehabilitation of the aging tower network, even if warranted, may not be feasible in light of decreasing capital budgets. However, by taking several steps toward evaluating the current health of the network and its associated performance during environmental loading events (wind, ice, seismic), available resources may be more effectively allocated and targeted toward maintaining an effective network. The global objectives of such an effort would include: 1) systematic assessment of the current physical conditions of the towers; 2) structural analysis of select towers under rationally designed environmental loading events; 3) development of a centralized electronic database for documenting and updating the physical properties and conditions of each tower. Structural rehabilitation and repair to towers identified as deficient could then be planned for and addressed by MoDOT as appropriate. The activities documented in this report are a Phase I effort toward meeting these global objectives. The following major tasks have been undertaken:

- Task 1 – Literature Review: Reviewed existing methodologies for function-based condition assessment of civil infrastructure. Reviewed historical failure mechanisms in free standing and guyed communications towers. Identified and reviewed appropriate design standards and dynamic analysis procedures.

- Task 2 – Condition Indexing System Development: Developed a rational "condition indexing" (CI) system that may be used to systematically quantify the physical condition of individual towers in the network. Output from the indexing system is a number ranging from 0 to 100 that may be used to prioritize mitigation and repair operations and to allocate resources accordingly.

- Task 3 – Condition Indexing System Demonstration: Demonstrated use of the condition indexing system for towers located at Taum Sauk (Iron County) and Ashland (Boone County). The Taum Sauk and Ashland towers were selected to represent towers in relatively poor physical condition and relatively good physical condition, respectively.

- Task 4 – Dynamic Structural Analyses of Key Towers: Conducted detailed structural analysis of the Taum Sauk (guyed) tower under seismic, wind and ice loading. Conducted detailed structural analysis of the Kansas City (free-standing) tower under wind and ice loading. A parametric study was conducted to evaluate the effects of simulated deterioration (e.g., corrosion) of the Taum Sauk tower. The Kansas City tower was evaluated in terms of performance relative to as-built and current design codes.

- Task 5 – Development of Centralized Electronic Database: Digitized relevant information regarding the tower network and the individual towers (e.g., location, appurtenances, structural drawings, condition index, etc.) in the form of a centralized, web-based electronic format.

1.3 Structure of Report

This report provides detailed documentation for each of the major tasks described above. Chapter 2, *Condition Indexing System*, describes the general rationale and methodology used to develop the CI system, outlines specific procedures for implementing the system, and demonstrates the system using the Taum Sauk and Ashland towers. Chapter 3, *Dynamic and Structural Modeling*, summarizes the procedures and results obtained from dynamic and static structural analyses conducted for the Taum Sauk and Kansas City towers. Chapter 4, *Database Development*, illustrates the centralized electronic data base. Finally, Chapter 5, *Summary, Conclusions, and Recommendations,* summarizes the report and includes specific recommendations for implementing the results and products of the project. Salient results from the literature review are incorporated at appropriate locations throughout the report.

2. Condition Indexing System

2.1 Introduction

A condition indexing (CI) system is a methodology or set of rules that may be used to systematically define the physical condition of a facility or network of related facilities. The output of a CI system is a quantitative condition index, or number, typically between 0 and 100. The lowest possible index (CI = 0) represents the “worst” condition possible for the facility. The highest possible index (CI = 100) represents the “best,” or ideal, condition.

A variety of CI systems have been implemented by state and federal agencies responsible for managing complex infrastructure networks made up of numerous similar facilities or structures. Notable CI applications include, for example, those developed by the U.S. Army Corps of Engineers (USACE) for managing paved road networks, shore protection structures, and earth dams (*e.g.*, Andersen and Torrey, 1995; Andersen et al., 1999a, 1999b, 2001).

In each of these cases, a rational ranking procedure is used to quantify the physical condition of the individual components comprising the larger, more-complex system (e.g. the guy wires comprising a guyed communications tower). Qualitative and quantitative parameters are defined that may be observed and recorded during site inspections (*e.g.*, corrosion of the guys, paint loss on the central mast, cracking of soil around the foundation, etc). Each component is assigned a quantitative value based on these observations to represent the physical condition of that particular component and is then weighted to capture the relative importance of that component to the overall health and performance of the structure. Weighted condition values for all of the system components are summed to generate an overall condition index for the facility. Overall condition may also be weighted by the severity of anticipated environmental loads at the structure’s location (e.g., seismic, wind, ice) and by the relative importance of that particular structure in the performance of the overall network (e.g., the number of communication channels linked to a particular radio tower). The output from the CI system is a numerical value that reflects the structure’s level of deterioration or loss of functionality, which may in turn be used as a rational basis for recommended action and a corresponding basis for the managing agency to allocate funds for repair, evaluation, maintenance, and rehabilitation (REMR) activities.

A condition indexing system has been developed to systematically quantify the physical condition of guyed communications towers within the MoDOT radio system network. The proposed CI system is a simplified version of the more complex CI system developed by Andersen and Torrey (1995) for USACE earth dams. The following sections of this chapter describe how the radio tower CI system was developed and how it may be implemented. Section 2.2 describes the general rationale and methodology by which the CI system was developed. A condensed discussion of the Andersen and Torrey (1995) CI system for earth dams is included to illustrate the basic methodology used to develop the system. Section 2.3 then describes specific procedures for implementing a proposed CI system for guyed radio towers. Section 2.4 demonstrates use of the proposed CI system for two towers in the MoDOT network. These include towers at the Taum Sauk and Ashland sites (Figure 1.1), which were selected to represent towers in relatively “poor” and relatively “good” condition, respectively.

2.2 General Rationale and Methodology

Although numerous types of CI systems exist (e.g., Hudson, 1992), many adopt the same general approach - The structure one wishes to assess (e.g., a communications tower) is subdivided into several subunits (e.g., central mast, foundation, guy wires); the condition of each subunit is rated; and the subunit ratings are combined in a rational and systematic manner to compute an overall condition of the structure as a whole.

Andersen and Torrey (1995) describe several steps required to develop a "function-based" conditioning indexing system. The CI system described in this report for communication towers is based on the following simplified synthesis of those steps. Each is described in more detail in the following sections within the context of the Andersen and Torrey (1995) CI system proposed for USACE earth dams.

1) Identify the functional components of the system.

2) Develop a component interaction matrix.

3) Code the interaction matrix to represent the strength of each interaction.

4) Define ranges between ideal and failed conditions for each component.

5) Develop weighting factors and formulate condition index scalar.

Appendix B contains a detailed description of each of these steps. Step-by-step CI development procedures are described in the context of the Andersen and Torrey (1995) CI system for earth dams.

2.3 Condition Indexing System for Guyed Communication Towers

A function-based condition indexing system for guyed communication towers has been developed following the general rationale and methodology described by Andersen and Torrey (1995) for earth dams (Appendix B). The overall assessment objective that was selected to form the basis for the indexing system is for the tower to maintain structural integrity and the ability to support communication in the time of emergency (e.g., following a seismic event). Each step in the CI development process is briefly described below.

Step 1: Identify the functional components of the system

Figure 2.1 is a schematic diagram of the basic components of a guyed communications tower. This includes: 1) a series of guys and associated guy anchoring systems, 2) the central mast, 3) the mast foundation system, and 4) environmental loading. The former three of these components are physical or "functional" components that may be directly assessed during inspections. Environmental loading is a "total system" component considered to consist of wind loading, ice loading, and seismic loading.

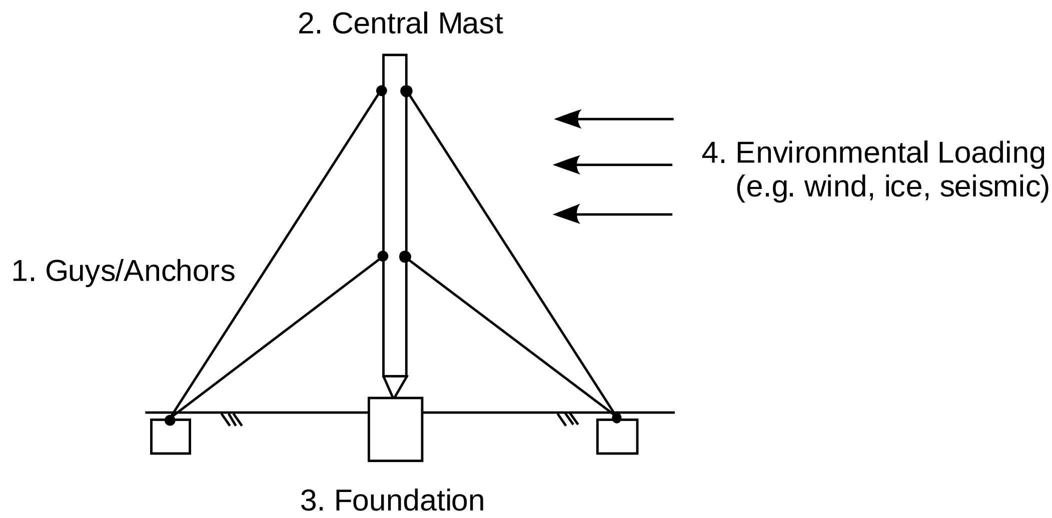


Figure 2.1. Four principal components of a guyed communications tower.

Step 2: Develop a component interaction matrix

Interaction matrices have been developed to qualitatively and quantitatively describe the interactions between the three physical tower components identified in Step 1, their relevant sub-components, and their surrounding environment. Figure 2.2 is a generalized 4 × 4 interaction matrix for the four-component tower system. The four principal components are represented in the diagonal cells of the matrix. Component interactions are described in the off-diagonal cells using the clockwise convention introduced previously.

Figure 2.3 is a more detailed 7 × 7 interaction matrix where several of the principal components have been divided into relevant sub-components. Here, the guy and guy anchor component has been subdivided into sub-components for the guy cables themselves and the guy anchoring systems. The latter, which is illustrated by the series of photographs in Figure 2.4, includes the anchor foundation block (typically a buried concrete block), the anchor rod (both above and below the ground surface), the anchor heads (gusset plates), the guy-to-anchor connections (e.g., cable wrap), the guy tensioning system, and, if present, any corrosion control system (e.g., cathodic protection). As also indicated on Figure 2.3, environmental loading has been divided into sub-components for (1) dynamic environmental loading, (2) static environmental loading, and (3) precipitation loading. The “dynamic” loading cell is intended to include dynamic loads from either wind (e.g., vortex shedding) or earthquakes. The “static” loading cell is intended to include so-called static environmental loads, primarily resulting from the build up of ice on the various tower components. Finally, the “precipitation” loading cell includes loading from weather related events such as rainfall, freeze/thaw cycling, and soil saturation/desiccation cycling. Table 2.1 describes detailed qualitative interactions between each of the functional and total system components following the clockwise interaction convention.

	COLUMN 1	2	3	4
ROW 1	**Guys and Guy Anchors**	relative tension governs twisting moment	total tension governs axial foundation load; relative tension governs lateral load	guy cross section governs extent of ice and wind load
2	height of mast governs number of guy stay levels; outriggers form direct connection with guys	**Central Mast**	height of mast governs axial foundation load	size of mast members govern ice and wind load
3	foundation settlement affects guy tension	foundation settlement affects mast orientation	**Foundation**	
4	guy oscillation from wind and seismic loading; loading from ice; fatigue; corrosion	fatigue and static force from wind and ice loading; corrosion; paint loss	freeze/thaw soil movements; soil dessication; erosion; seismic liquefaction	**Environmental Loading**

Figure 2.2. 4 × 4 interaction matrix representing guyed communication tower system.

	Column						
Row	**1**	**2**	**3**	**4**	**5**	**6**	**7**
1	Guys	3	3				
2	2	Guy Anchors					
3	2		Mast	1			2
4	1	1	1	Foundation			
5	3	1	3	2	Dyn. Env. Loading		
6	2	1	2	2	2	Static Env. Loading	
7		2	2	2			Precip. Loading

Figure 2.3. Total system interaction matrix for assessment of overall tower structural integrity.

(a)

(b)

(c)

(d)

Figure 2.4. Components of guy anchor system: (a) guy anchor showing gusset plate and anchor rod extending into ground surface; (b) detail of gusset plate; (c) detail of tensioning system; (d) detail of cable-to-anchor connection.

Step 3: Code the interaction matrix to represent the strength of each interaction

The numerical values in the off-diagonal cells of the interaction matrix (Figure 2.3) designate the relative strength of the interactions between the functional and total system components of a guyed tower. Interaction strengths were coded from zero to four using Hudson's (1992) ESQ interaction levels (Appendix B, Table B.3). Tower design criteria and literature review of historical damage and failure mechanisms (e.g., Madugula, 2002) were also considered to assist in assigning several of the interaction strengths.

Table 2.1. Interaction descriptions for maintaining overall structural integrity of guyed communications tower.

Row	Column	Interaction Description
1	2	corrosion in guy can proceed to anchor; excess tension can overstress anchor; guy oscillations can lead to fatigue in guy/anchor connection
1	3	excess tension or slack in guy produces twisting moment in mast (stress on bracing members); overstress in guys increases axial load in mast members; guy rupture induces dynamic loads in mast and increases static forces; variations in guy tension alters dynamic response of mast
2	1	movement of anchor may cause slackening in guy
3	1	excess twist, out of plumb, or loose members can cause excess tension/slack in guys; rocking of mast can lead to galloping in guys.
3	4	excess twist, out of plumb increases lateral loads to foundation
3	7	clogged drain holes can lead to corrosion from precipitation
4	1	movement and settlement can lead to changes in guy tension
4	2	movement or settlement relative to anchor foundations may lead to differential guy stress; overturning or rocking can lead to excess anchor stress
4	3	movement or settlement can alter mast orientation
5	1	wind and seismic loads introduce oscillations leading to potential overstress; fatigue
5	2	multiple support excitation can lead to differential guy tension
5	3	wind loading (vortex shedding) causes stresses in mast members; fatigue; base shear from seismic loading
5	4	seismic loads can lead to foundation soil liquefaction; seismic induced settlement; wind loads transmitted as lateral load
6	1	build up of ice can increases static tension, changes natural frequency
6	2	ice sliding down guy can damage guy/anchor connection (LeBlanc, 1988)
6	3	build up of ice increases gravity load; falling ice may induce significant dynamic vibrations
6	4	build up of ice increases gravity load
6	5	build up of ice increases wind load from increased member cross section (radial accumulation); build up of ice changes reactive mass of mast leading to modified dynamic response
7	2	precipitation may soften anchor foundation soils; freeze/thaw and desiccation loading may cause movement of anchor foundation
7	3	precipitation may lead to corrosion
7	4	precipitation may soften foundation soils; freeze/thaw and desiccation loading may cause movement of foundation

Step 4: Define ranges between ideal and failed conditions for each component

In order to develop an overall condition index for a particular tower, the physical condition of the functional components must be individually ranked from 0 to 100. These component rankings are then weighted by considering "cause" and "effect" scores for each functional component.

The functional components (guys, guy anchors, foundation, and central mast) are ranked based on observed deviation from the ideal condition. Ideal conditions have been designed to include observations for the condition of items complying with Annex J (Maintenance and Condition Assessment) of Standard ANSI/TIA/EIA-222-G. These include the structural condition of the central mast, finish (e.g., paint condition), lighting, grounding, antennae and lines, appurtenances (safety, climbing facilities, etc), guy cables, foundations, and anchors. Proposed definitions for corresponding ideal conditions of the four function components are provided in Table 2.2. Each of these components should be assigned a value from 0 to 100 following Table 2.3 based on the amount of observed deviation from the ideal conditions defined in this manner.

Table 2.2. Definitions of ideal and failed conditions for functional components guyed communication tower (ideal conditions based on consideration of ANSI/TIA/EIA-222-G).

System Subunit	Definition of Ideal Condition
Guy Cables	No cut or missing guy cables
	No visible corrosion, breaks, nicks, or kinks
	Guy tension well within design tolerance
	No significant deviations in guy tension for given stay level
Guy Anchors	No visible corrosion of anchor rod above or below ground surface (inspection requires temporary excavation)
	Anchor heads (gusset plates) clear of ground surface and corrosion
	No visible settlement of anchor blocks
	No visible cracking or heaving of earth surrounding anchor block
	Corrosion control measures in place (if applicable)
	No excessive growth of vegetation around anchor
	Cable connectors secure
	Cable clamps properly applies and bolts tight
	Cable wraps properly and fully wrapped
	Poured sockets secure and showing no separation
	Shackles, bolts, pins, and cotter pins secure
	Tensioning device free of corrosion, bending
Central Mast	No damaged, loose, or bent members
	No missing members
	No loose or missing bolts and/or nut locking devices
	No flaking of paint or loss of galvanization
	No visible corrosion or pitting of members
	No water collection in members (clogged drain holes)
	Plumb and twist within tolerance (ANSI/TIA/EIA-222-G, Annex J)
Foundation	No visible settlement or lateral movement
	No visible cracking, spalling, chipping, or splitting in concrete
	No visible erosion or undermining of foundation soil
	No visible corrosion of mast/foundation connection
	No standing water on foundation or surrounding soil; no low spots to collect standing water
	No excessive growth of vegetation around foundation

Table 2.3. Indexing scale for quantifying condition of system components.

Condition Index	Condition Description
85 – 100	Excellent: No noticeable deviation from ideal condition
70 – 84	Very Good: Only slight deviations from the ideal condition are evident
55 – 69	Good: Some deviation from the ideal condition evident but function is not significantly affected
40 – 54	Fair: Moderate deviation from the ideal condition evident but function is adequate
25 – 39	Poor: Serious deviation from ideal condition in at least some portion of the component; function is inadequate
10 – 24	Very Poor: Extensive deviation from ideal condition: Component is barely functional
0 – 9	Failed: All or a potion of component is missing or has failed

Step 5: Develop weighting factors and formulate condition index scalar

An expression for the overall condition index of a guyed tower may be developed by considering the "cause" and "effect" scores for each functional component in the interaction matrix. Table 2.4 summarizes these scores and computes the weighting factors for each functional component.

Table 2.4. Development of weighting factors from guyed tower interaction matrix.

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	---	---

Note that of the four functional components, the guy cables and central mast are the most interactive within the total system (highest weighting factors). The guy anchors and foundation are the least interactive (lowest weighting factor). An overall condition index may be computed using these weighting factors as

$$CI_{gt} = CI_{gc}(0.31) + CI_{ga}(0.19) + CI_{cm}(0.31) + CI_{fd}(0.19) \tag{2.1}$$

where CI_{gt} is the overall condition index of the guyed tower, CI_{gc} is the component condition index of the guy cables, CI_{ga} is the component condition index of the guy anchors, CI_{cm} is the component condition index of the central mast, and CI_{fd} is the component condition index of the foundation. The overall condition index for the tower ($0 < CI < 100$), may be correlated to a qualitative description and recommended action as summarized in Table 2.5.

Table 2.5. Proposed 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	

2.4 Demonstration of Condition Indexing System

Procedures for conducting a condition assessment using the proposed CI system may be summarized as follows:

1) Rank the physical condition of each principal component (guys, guy anchors, central mast, and foundation) from 0 to 100 based on the observed deviation from ideal conditions using Table 2.2. This produces four numbers from 0 to 100, including CI_{gc}, CI_{ga}, CI_{cm}, and CI_{fd}.
2) Compute the overall condition index for the tower (CI_{gt}) by applying weighting factors using equation 2.1.
3) Correlate the overall condition index to a qualitative description and recommended action using Table 2.5.

Use of the proposed CI system is demonstrated in the following by considering two towers in the MoDOT network. These include towers located at Taum Sauk (Iron County) and Ashland (Boone County) (see Figure 1.1). The Taum Sauk and Ashland towers were selected to represent towers in relatively poor physical condition and relatively good physical condition, respectively.

2.4.1 Condition Index of Taum Sauk Tower

The Taum Sauk tower site was visited on December 7, 2004 for visual inspection and condition indexing following the procedures described above. Figures 2.5 through 2.8 show photographs of the overall tower and details of the functional components considered for inspection.

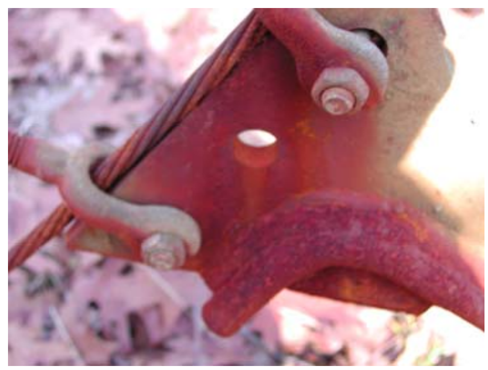
Table 2.6 summarizes corresponding condition indices assigned to each functional component using Tables 2.2 and 2.3. The overall CI for the Taum Sauk tower based on equation 2.1 is 54, which, following Table 2.5 corresponds to "Fair: Moderate deterioration but function is still adequate." Economic analysis of repair alternatives is recommended, which may guided in part by considering the corresponding changes to condition index. If, for example, the guy cables were completely replaced with new cables (CI_{gc} = 100), the overall CI would increase to 70, or good/very good. The significant increase in CI reflects the dominance of the guy cable condition on the overall health of the multi-component structure.

Table 2.6. Summary of condition indices assigned for Taum Sauk tower.

Subunit	CI	Weighted CI
Guy Cables, CI_{gc}	50	15.5
Guy Anchors, CI_{ga}	50	9.5
Central Mast, CI_{cm}	60	18.6
Foundation, CI_{fd}	55	10.5
Total		**54**

(a) (b)

Figure 2.5. Photographs of Taum Sauk tower: (a) entire tower; (b) upper stay levels.



(a) (b)

(c) (d)

Figure 2.6. Details of guy and guy anchor condition for an anchor on Taum Sauk tower: (a) significant corrosion of guy cable at cable/anchor connection; (b) and (c) corrosion staining on gusset plate; (d) corrosion on tensioning system.

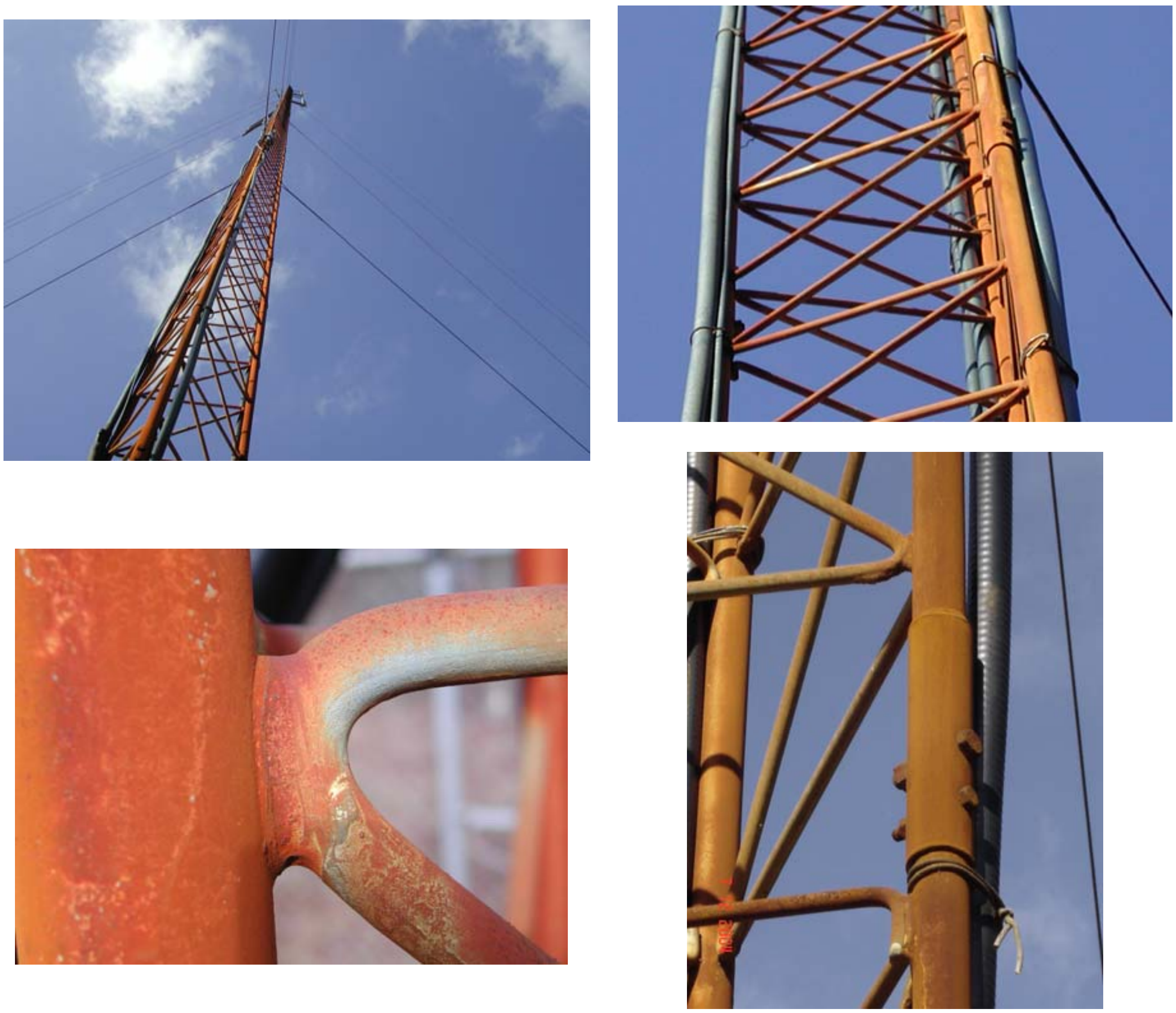


Figure 2.7. Various details of central mast of Taum Sauk tower showing corrosion and staining.

Figure 2.8. Various details of Taum Sauk foundation and foundation/mast connection.

2.4.2 Condition Index of Ashland Tower

The Ashland tower site was visited on April 12, 2005 for visual inspection and condition indexing. Figures 2.9 through 2.13 show photographs of the overall tower and details of the functional components considered for inspection.

Table 2.7 summarizes corresponding condition indices assigned to each functional component. The overall CI for the Ashland tower is 85, which corresponds to "Excellent: No noticeable defects; some aging or wear may be visible." No immediate action is warranted.

Table 2.7. Summary of condition indices assigned for Ashland tower.

Subunit	CI	Weighted CI
Guy Cables, CI_{gc}	90	27.9
Guy Anchors, CI_{ga}	80	15.2
Central Mast, CI_{cm}	80	24.8
Foundation, CI_{fd}	90	17.1
Total		**85**

(a)

(b)

Figure 2.9. Photographs of Ashland tower: (a) lower stay levels; (b) upper stay levels.

Figure 2.10. Detail of Ashland guy cable showing minimal corrosion.

Figure 2.11. Details of Ashland cable anchors.

Figure 2.12. Details of Ashland central mast. Significant paint flaking is observed but minimal loss of galvanization.

Figure 2.13. Details of Ashland foundation and shear pin.

2.5 Weighting Factors

The main input parameters included in the proposed CI system are related directly to physical conditions. Specifically, this includes the condition of the guys, guy anchors, central mast, and foundation. In addition to these condition-related parameters, however, there are numerous other more subjective factors that may also be considered to contribute to an overall index applicable for making maintenance and resource allocation decisions. Most notably, this includes the relative importance of an individual tower in the overall MoDOT network.

Considering these additional types of subjective parameters provide a more comprehensive basis for condition indexing. For example, if a select tower is determined to be in relatively poor physical condition but is not considered relatively important to the overall health of the radio tower network (i.e., the state-wide system of multiple towers), then the priority for the repair of this tower may be relatively low. If, on the other hand, a tower is determined to be in fair condition but the tower is also considered critical to the overall health of the network, then the priority for the repair of the tower may be relatively high.

Weighting factors may be used to adjust the physical condition indices determined using the procedures described in the previous sections to account for these somewhat more subjective types of input variables. We suggest, for example, that the relative importance of a particular tower to the health of the overall network may be quantified by considering the number of adjacent towers for which that tower "serves" as a hub. Considering Figure 1.1, for example, the Taum Sauk tower serves as a hub to seven adjacent towers, which would suggest that the Taum Sauk tower is among the most important to the overall network. The Ashland tower, which also serves as a hub to seven adjacent towers, may also be considered relatively important. The Skidmore tower, on the other hand, serves as a hub to only two other towers.

If relative importance is quantified in this manner, then this subjective input variable may be considered in an objective sense by multiplying the physical CI of a tower by a weighting factor based on relative importance, such as those proposed in Table 2.8. The effect of the weighting factors is to reduce the magnitude of the physical condition index such that the CI of the most important towers are reduced by the greatest amount. If resources are allocated to towers with the lowest adjusted CI, then towers in the poorest physical condition and of the most importance to the network are prioritized. The CI of the Taum Sauk tower, for example, which was determined previously to have an unweighted value of 54 and following Figure 1.1. and Table 2.8 would have a weighting factor of 0.5, would be reduced to CI = (54)(0.5) = 27. Similarly, the Ashland tower (unadjusted CI = 85), would be adjusted to CI = (85)(0.5) = 43.

Table 2.8. Weighting factors based on relative importance of tower to overall network.

# of Adjacent Towers	Weighting Factor
2	1.0
3	0.9
4	0.8
5	0.7
6	0.6
7	0.5
8	0.4

3. Dynamic and Structural Modeling

3.1 Introduction

The primary threats of tower damage or structural failure come form three sources: excessive wind loading, ice loading, and seismic loading. Although many studies exist that have been able to model the effects of ice and wind loading, few studies have been performed that can effectively simulate the performance of free-standing or guyed towers during an earthquake.

Section 3.2 of this chapter provides a detailed summary of background information relevant to seismic and more general dynamic analysis of guyed and free-standing communications towers. Relevant design codes and procedures for simulating seismic loading events are summarized. Section 3.3 introduces the scope of the dynamic structural analyses conducted for specific towers in the MoDOT network as part of this project and summarizes the associated software used for the effort. Section 3.4 summarizes procedures and results for analysis of the guyed tower at Taum Sauk. This includes dynamic structural analysis under simulated seismic events using SAP200 software, as well as static structural analysis under wind and ice loading events using ERITower software. Results from a complimentary parametric study conducted to simulate aging and deterioration of the Taum Sauk tower and the associated effects on its performance are also summarized. Section 3.5 summarizes procedures and results for analysis of the free-standing tower at Kansas City. This includes static analysis under wind and ice loading. Section 3.6 closes the chapter with a list of observations and conclusions from the analyses.

3.2 Background

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 3.1) and guyed (Figure 3.2) 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” (ASCE 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 2001). 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 3.1 provides a partial list of several notable tower failures and corresponding failure mechanisms related to more general dynamic loading effects (predominantly wind related).

Figure 3.1. Example of a typical self-supporting (free standing) tower.

Figure 3.2. Example of a typical guyed mast tower.

Table 3.1. 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

3.2.1 Numerical Simulations

Self Supporting Towers

In 1994, Mikus conducted one of the first studies using numerical simulations (which many later works would be based off) 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 this 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, 1996). 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" (ADINA R&D 1987). 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 (1996) 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 3.2) were subjected to three different seismic excitations to determine if there were any similarities present in the dynamic tower response.

Table 3.2. Summary of tower characteristics used in Amiri (1996) 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 as Figure 3.3 was produced to predict total base shear in towers of varying heights.

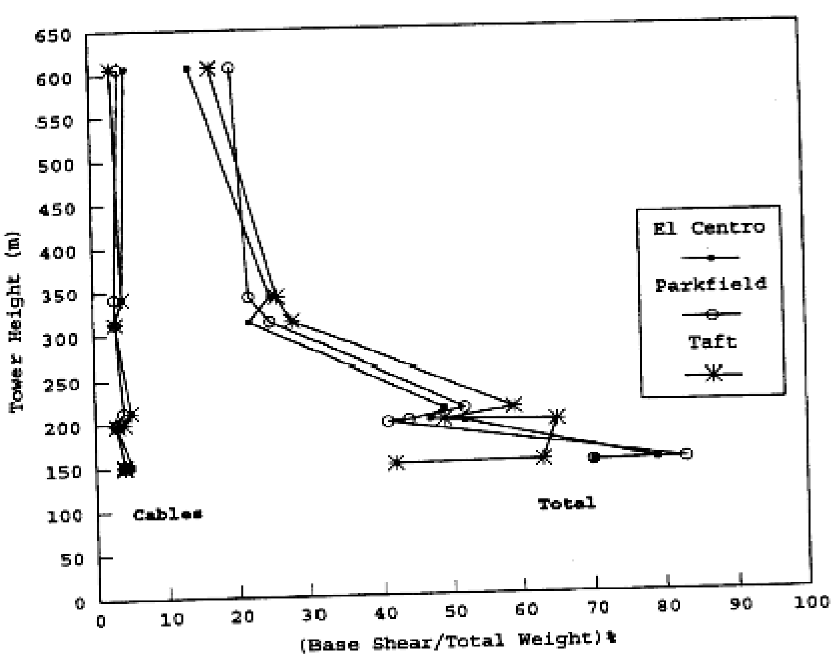


Figure 3.3**.** Base shear versus tower height (Amiri, 2002).

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 this 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 the upper clusters.

3.2.2 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. 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, McClure, Zaugg 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 and acceleration-to-velocity ratio (a/v) horizontal accelerograms. Results were plotted much like the work of Amiri 1996, 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 modal 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, McClure, and Zaugg; Khedr and McClure 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 off 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, McClure, and Zaugg. 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 this 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 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 (ASCE, 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.

3.2.3 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 3.3.

Table 3.3. 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.

3.2.4 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 3.4, 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:

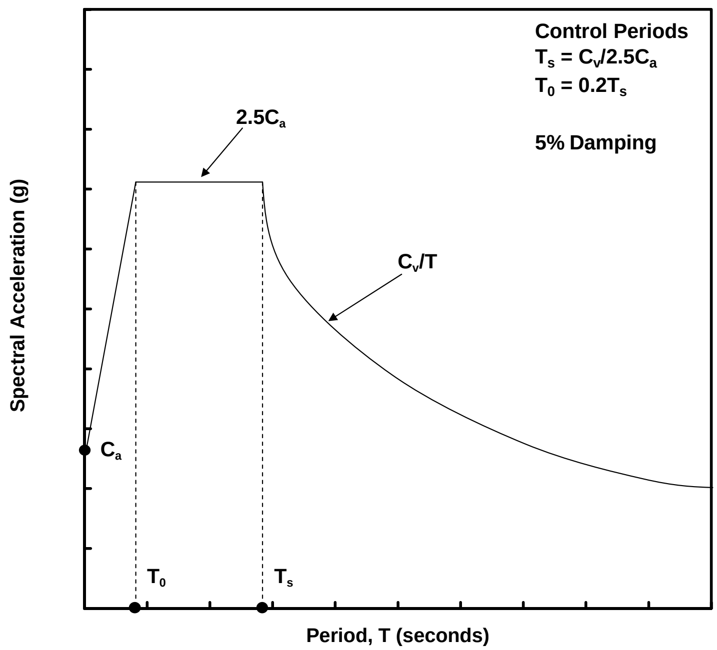


Figure 3.4. Uniform Building Code (UBC, 1997) response spectrum.

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 3.5 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 3.4. 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).

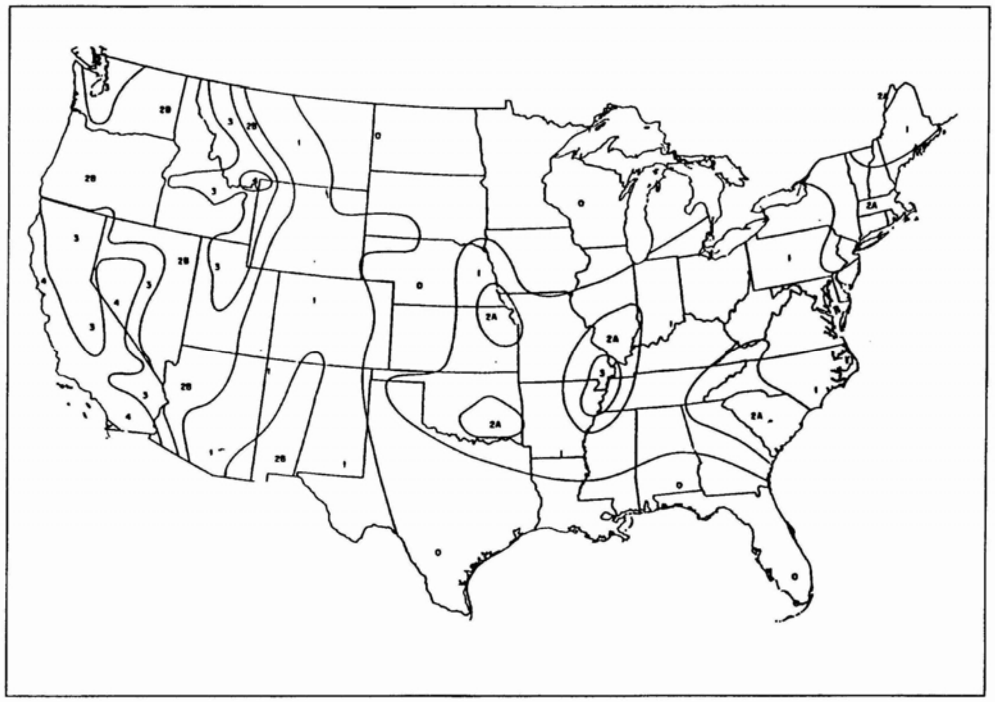


Figure 3.5. UBC seismic zones for the United States. Note: The contour intervals originating in Southeast Missouri are 3 and 2A. The majority of Missouri falls within contour 1. The small contour in Northwest Missouri is 2A.

Table 3.4. 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 3.5 and C_v may be determined from Table 3.6. Once the seismic coefficients are determined, the parameters T_s and T_0 defining the remainder of response spectrum can be calculated as

$$T_s = \frac{C_v}{2.5C_a}$$
$$T_0 = 0.2T_s \tag{3.1}$$

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 Appendix C.

Table 3.5. 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 3.6. 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

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

and the response spectrum is shown as Figure 3.6a. Figure 3.6b shows an equivalent time-domain accelerogram for a “short” duration (10 s) seismic event developed using procedures described in Appendix C. Figure 3.7a and 3.7b 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.

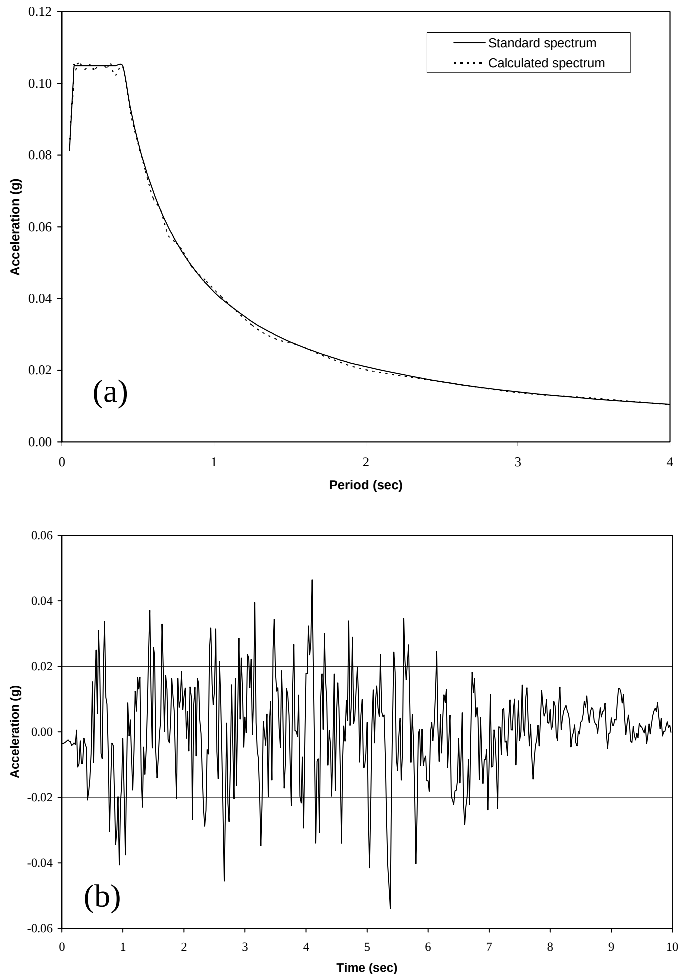


Figure 3.6. Response spectrum (a) and equivalent accelerogram (b) for short duration seismic event at Taum Sauk site (Rock, Zone 2A).

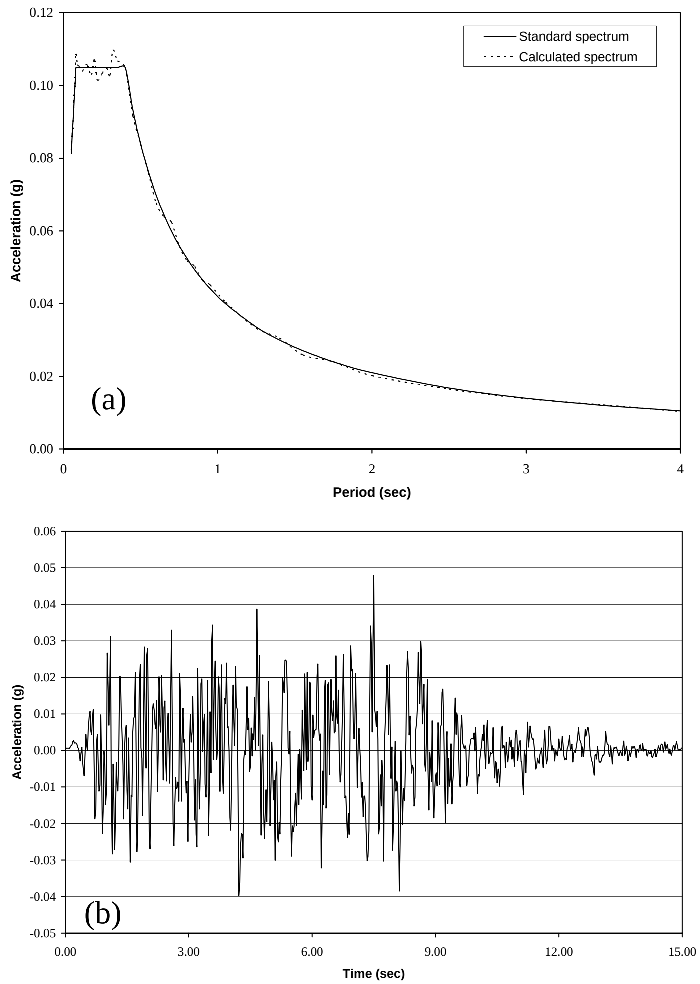


Figure 3.7. Response spectrum (a) and equivalent accelerogram (b) for long duration seismic event at Taum Sauk site (Rock, Zone 2A).

3.3 Scope of Dynamic and Structural Analysis

Response of the Taum Sauk (guyed) and Kansas City (free standing) towers under various forms of environmental loading (seismic, wind, ice) was predicted using computer software. Dynamic (earthquake) response was modeled using SAP 2000 v9.0, whereas the response under wind and ice loads was predicted using ERITower v3.0. The Taum Sauk tower was modeled for both earthquake loading and wind and ice loading, whereas the Kansas City tower was modeled for wind and ice loading combinations only. Detailed modal analysis and time history analysis of the Taum Sauk tower subjected to the earthquake loads defined previously were performed using non-linear modal analysis. A parametric study was conducted to simulate aging, deterioration, and corrosion of the Taum Sauk tower and the associated effects on its performance. Response of the Kansas City tower according to design standards TIA-222-C and TIA-222-F was evaluated to check for passing or failing performance under the as-built and current code requirements, respectively.

3.3.1 Description of 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 was 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.

3.3.2 Description of ERITower v3.0

ERITower v3.0 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 9^{th} 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.

3.4 Analysis of Taum Sauk Tower

The Taum Sauk tower (Figure 3.8) is a 150 foot, 3 sided, guyed tower located on Taum Sauk Mountain. Analysis was performed for earthquake loading using SAP software and for wind and ice loading using ERITower software. Results of the seismic modeling are presented first followed by the results of the wind and ice analysis.

3.4.1 Development of SAP Model

The tower was modeled using the information provided by structural tower drawings (Appendix D). 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 additional antennas) were not modeled in this analysis, since not 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 leg 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 3.9.

3.4.2 Loads

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 3.10 and 3.11 were specified for two different earthquake seismic events on rock profile. The standard spectrum function of Figure 3.10 was used to produce a normalized standard spectrum with respect to gravitational acceleration as shown in Figure 3.12.

The response spectrum function was applied in three directions: X, Y and Z (vertical) directions as shown in Figure 3.13 based on 16 modes from Eigenvector modal analysis. The time history functions in Figure 3.14 were used for the nonlinear time history analysis based on 16 modes from Ritz vector modal analysis.

3.4.3 Load Combinations

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.

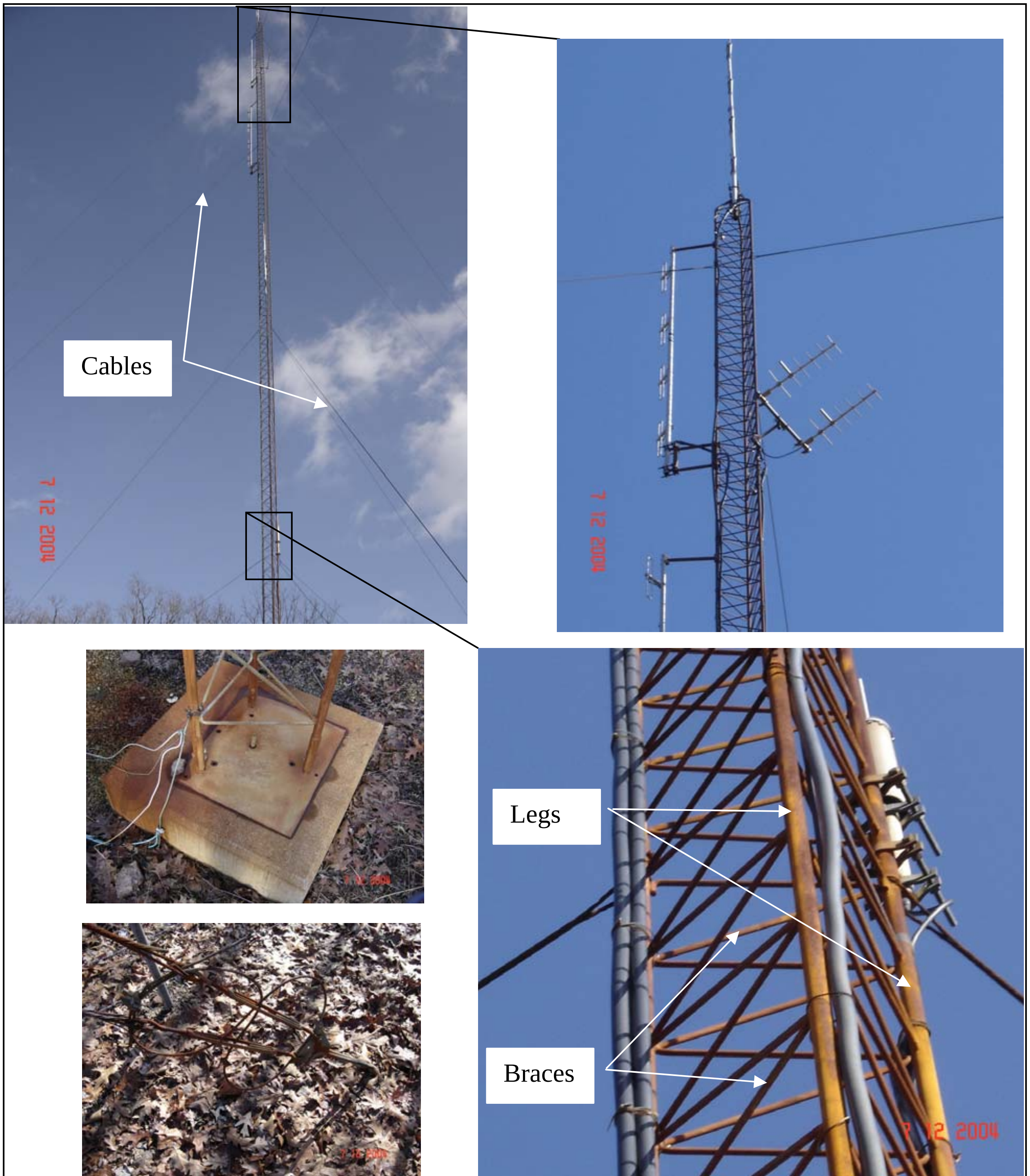


Figure 3.8. Key Elements of Taum Sauk Tower

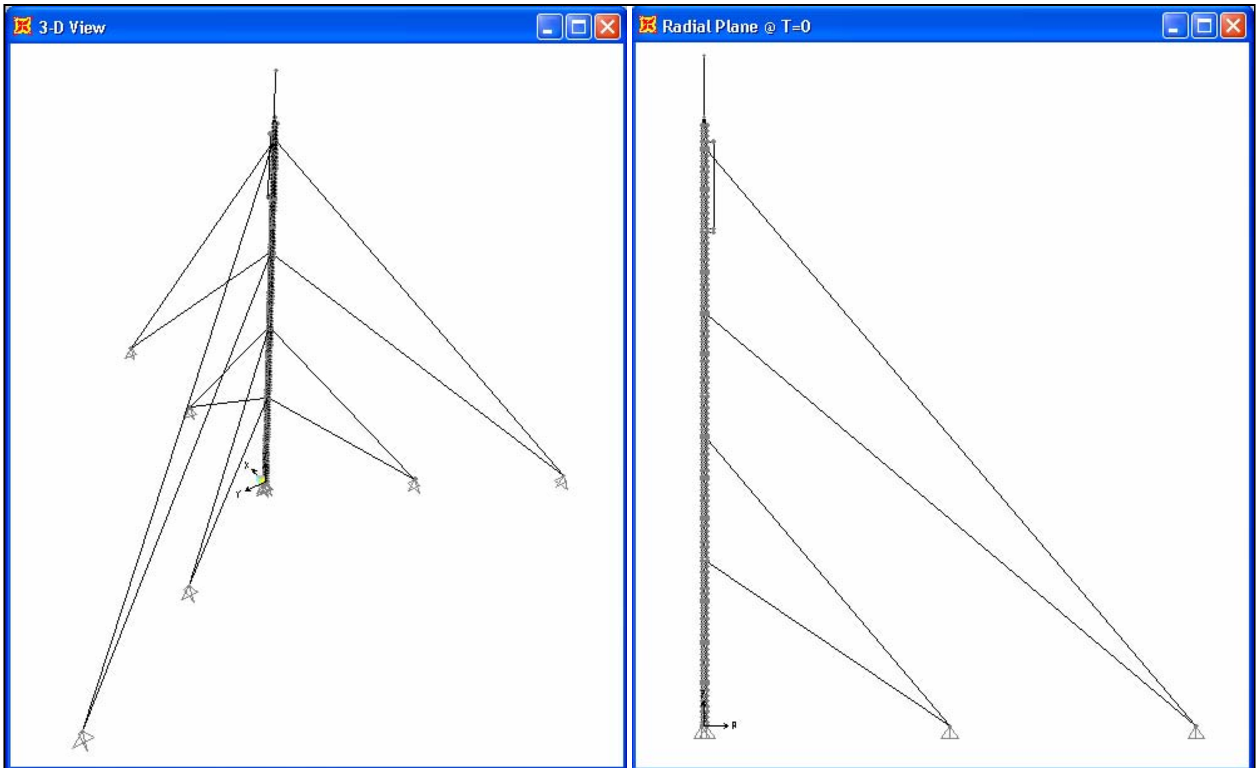


Figure 3.9. SAP model of Taum Sauk tower

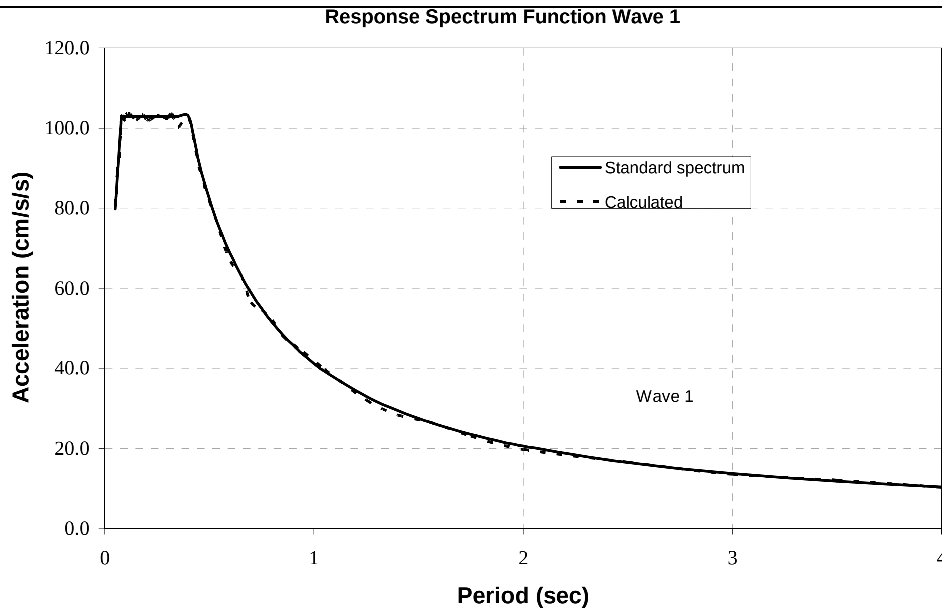


Figure 3.10. Response spectrum function for wave 1

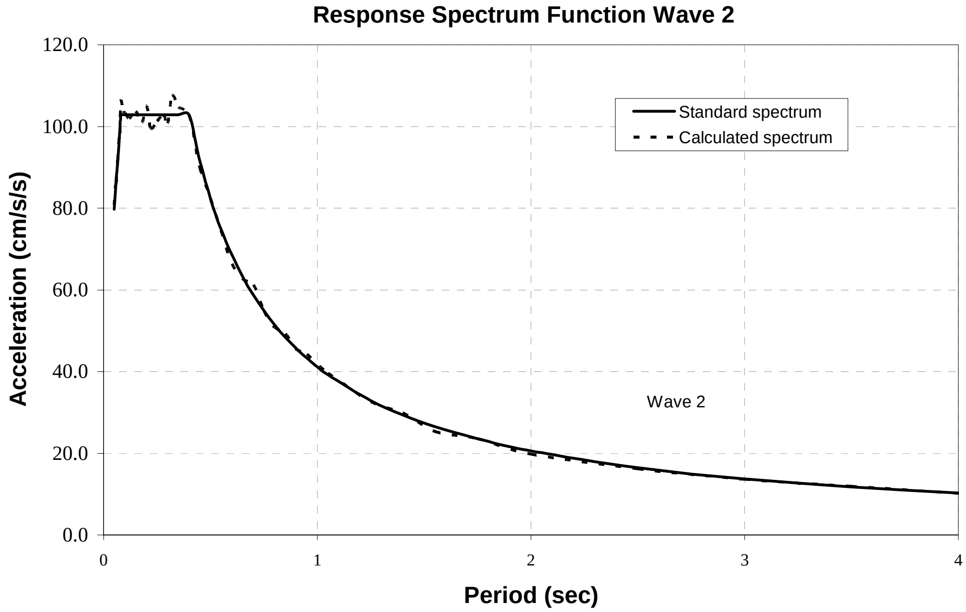


Figure 3.11. Response spectrum function for wave 2

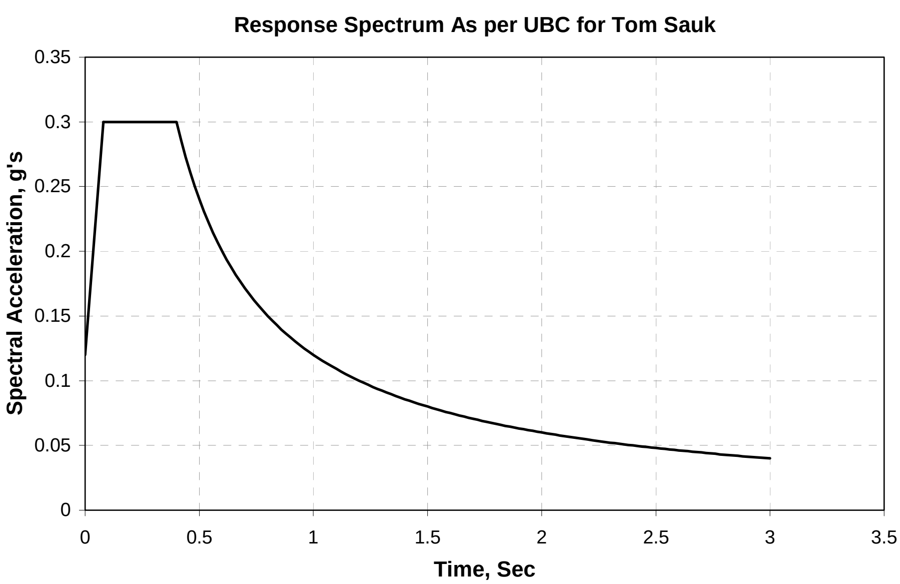


Figure 3.12. Standard Response Spectrum Function

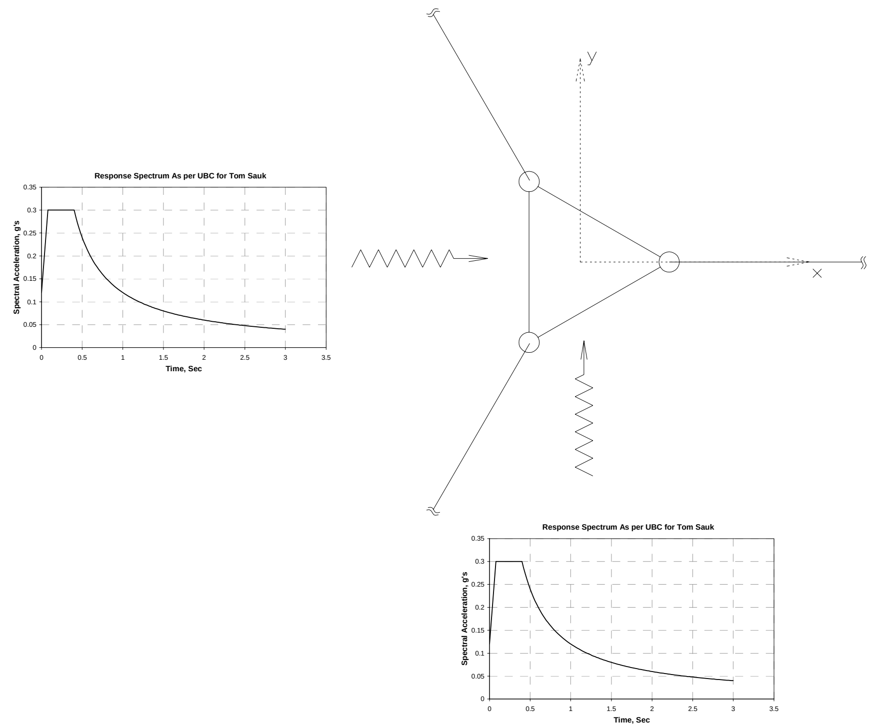


Figure 3.13. Load of Response Spectrum Function on the Taum Sauk Tower

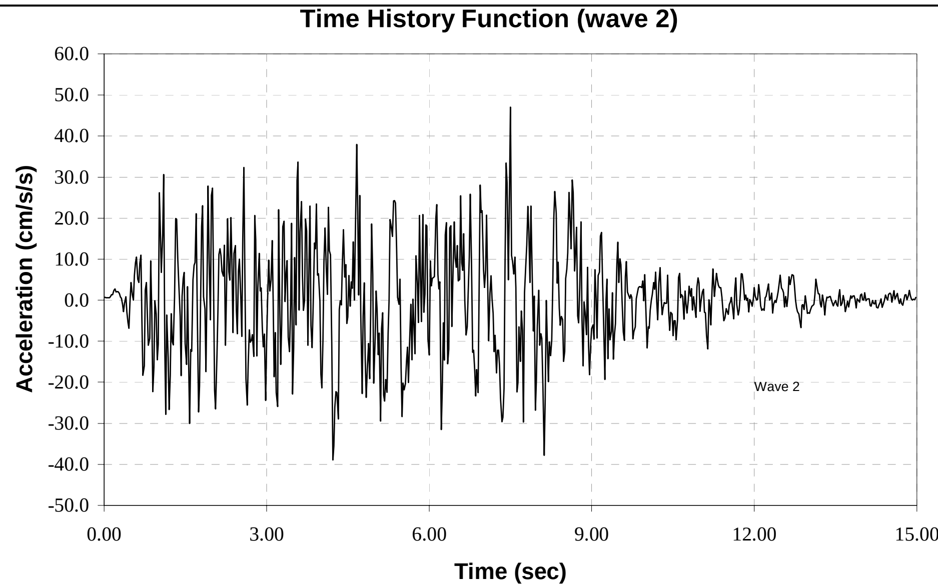


Figure 3.14. Time History Function (wave 2)

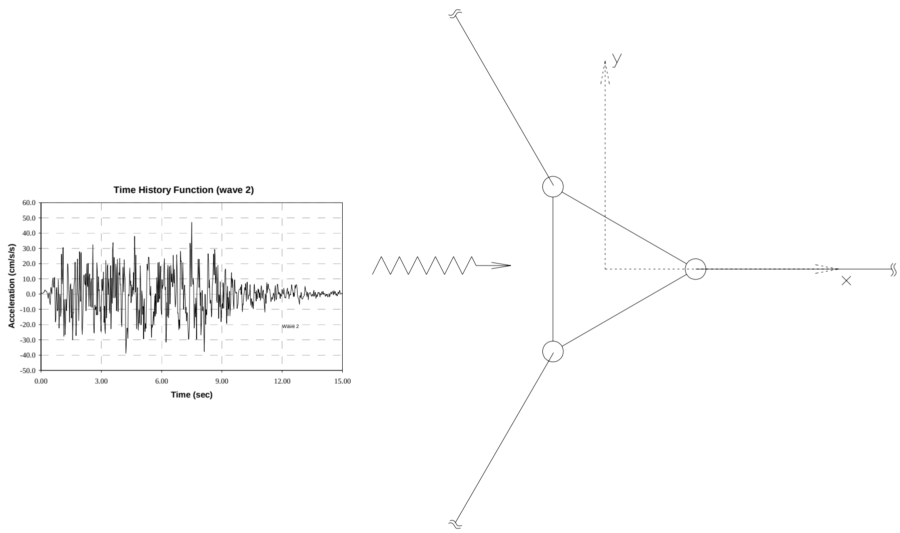


Figure 3.15. Loading of the time history function on the tower

3.4.4 Key Results from Dynamic Analysis

Mode shapes

A total of sixteen mode shapes were selected for the analysis. Selected modes are shown in Figure 3.16 and the remaining 16 modes are shown in Appendix E. The modal analysis was used for the dynamic analysis using the response spectrum loading function as described earlier.

Defining Critical Members

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 E. The maximum results from all combinations are shown in Figure 3.17, 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.

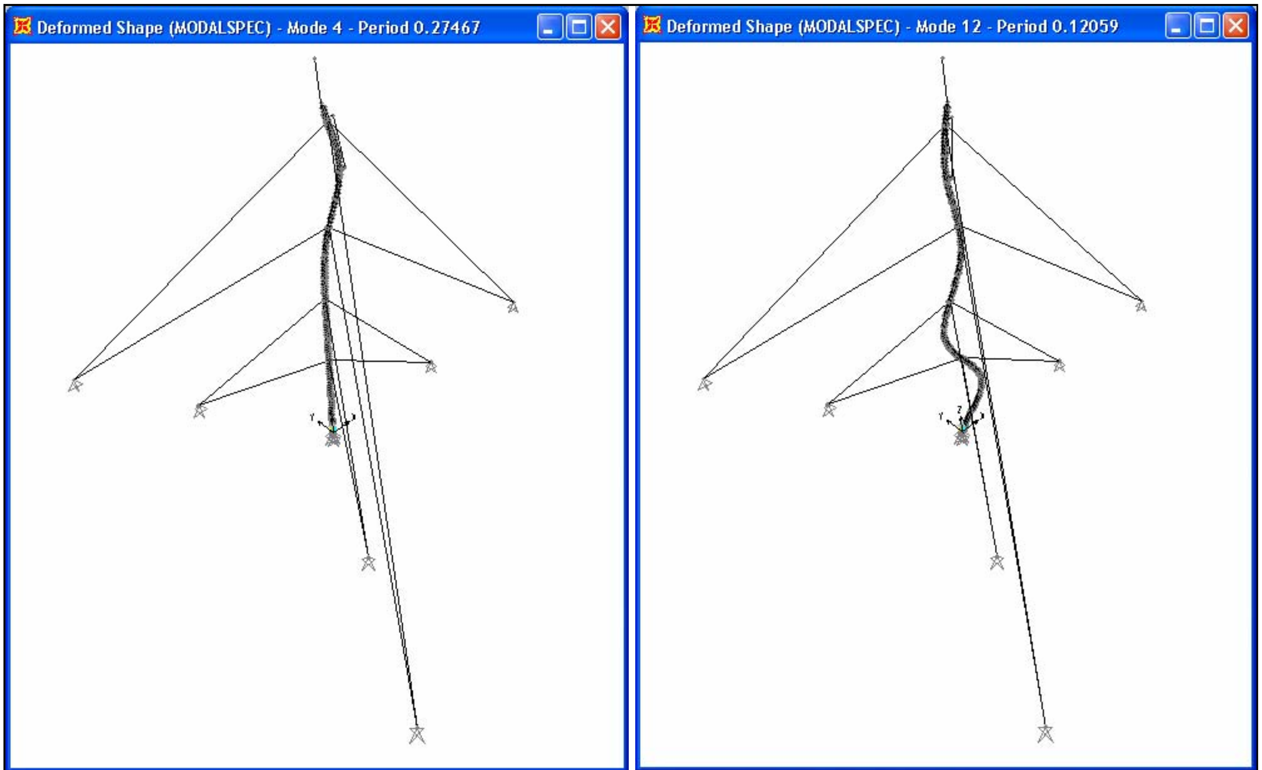


Figure 3.16. Selected mode shapes 4 and 12

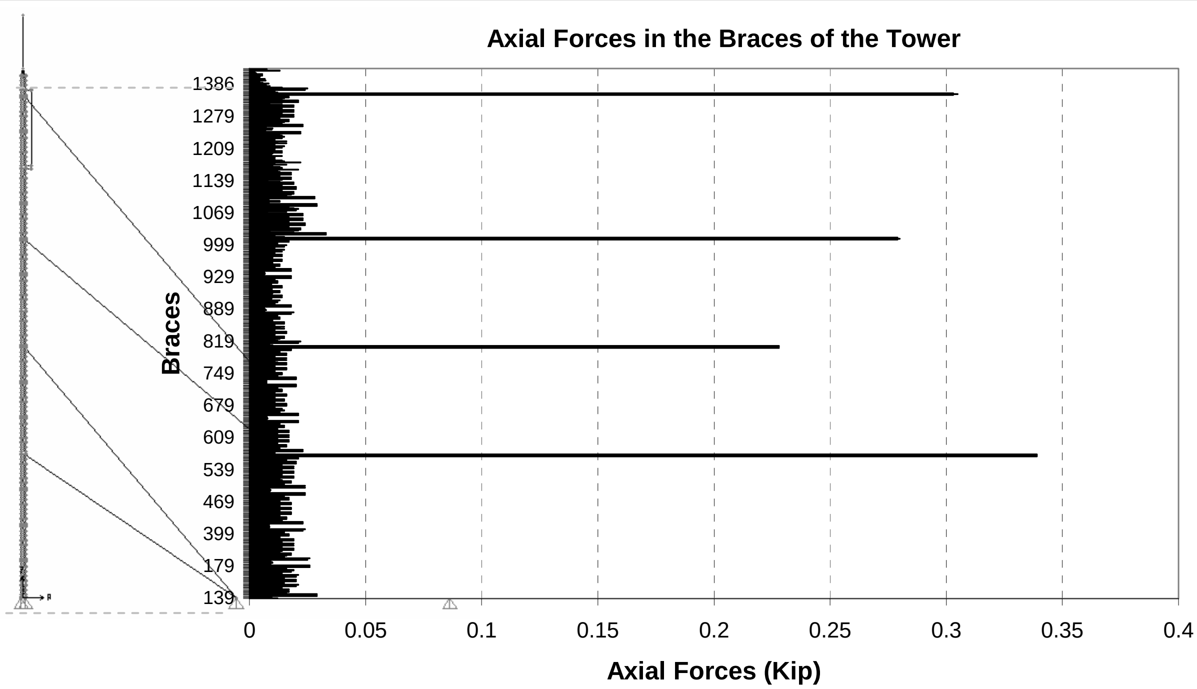


Figure 3.17. Axial Forces of Braces for Undamaged Structure of the Tower

3.4.5 Development of ERITower Model

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 grade of the guys was input as EHS. 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 from the drawings and input into the software. 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 3.18 shows a model of the tower.

3.4.6 Wind and Ice Loading Analysis

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.

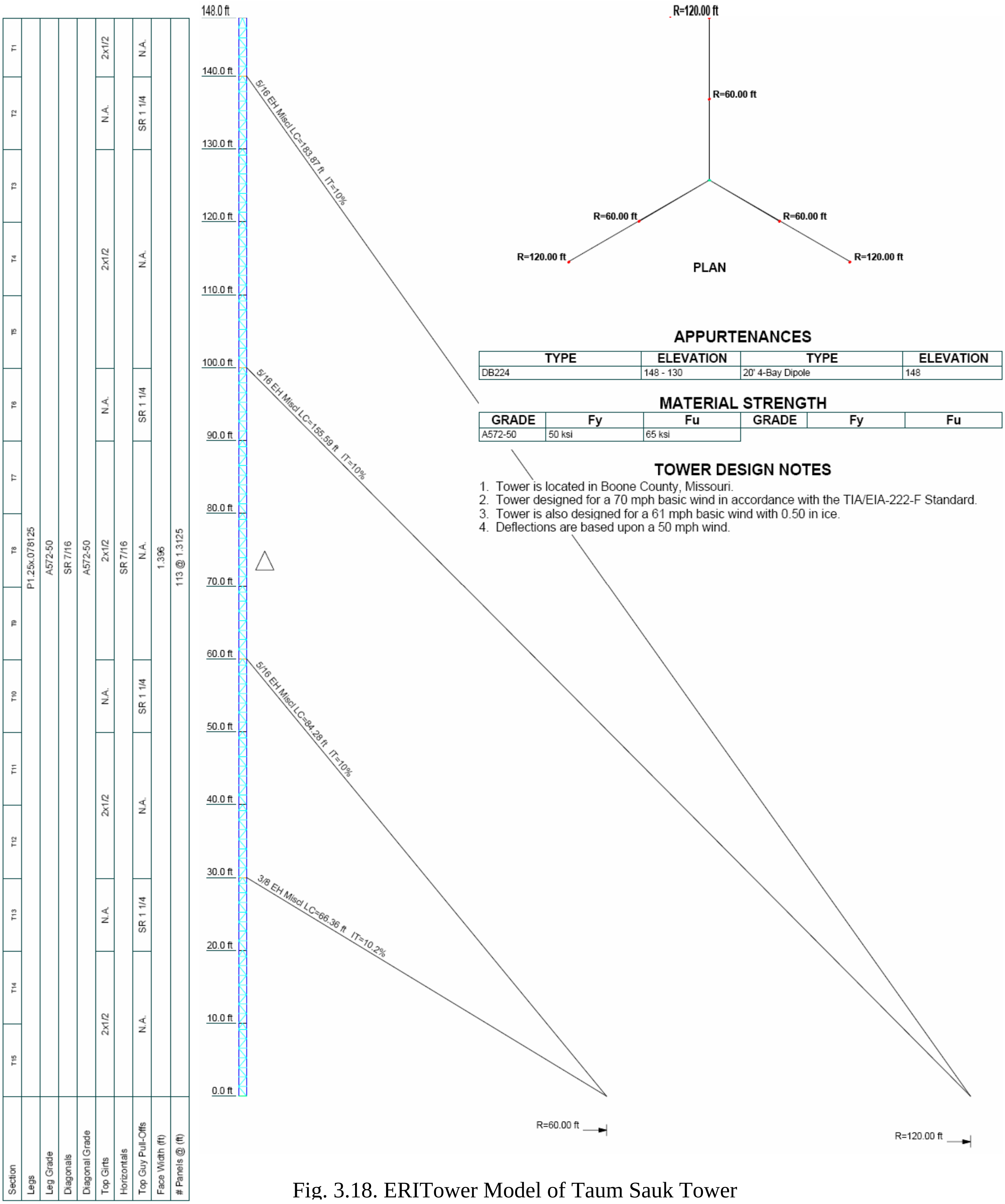


Fig. 3.18. ERITower Model of Taum Sauk Tower

After the 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.

$$\frac{Combined \quad Stress \quad Ratio}{Allowable \quad Stress \quad Ratio} \leq 1.333$$

From this check for 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 are tables 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 3.7 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 3.7. 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

3.4.7 *Parametric Study*

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. The wind and ice loading analyses using ERI Tower software is presented first followed by the seismic loading results using SAP software.

The 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.

The tower rating as well as the component capacities are given in Tables 3.8 – 3.10 and graphically represented in Figures 3.19 – 3.21. 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 Table 3.9a exhibit somewhat irregular behavior, which is likely a result of significant overstressing after 10% damage to the bracing is reached. Results in Table 3.9b, 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%.

Table 3.8. 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

Table 3.9a. 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 3.9b. Bracing Damage Effect (0 – 9%)

Bracing Damage %	Tower Rating %	Guy Capacity %	Leg Capacity %	Bracing Capacity %
0	81.7	81.7	61.9	70
5	87.3	81.6	61.5	87.3
6	92.1	81.5	61.4	92.1
7	98	81.5	61.3	98
8	106.5	81.5	61.2	106.5
9	123.9	81.5	61.1	123.9

Table 3.10. 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

Additional observations include the following:

- The diagonal braces are the most stressed members in the tower structure at a 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 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 various 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% every 10% increments. Four different cases of damage were considered (Table 3.11). 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 3.11. Tower member sizes for the 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
Diameter (d2)	0.026	0.0234	0.0208	0.0182	0.0156	0.013

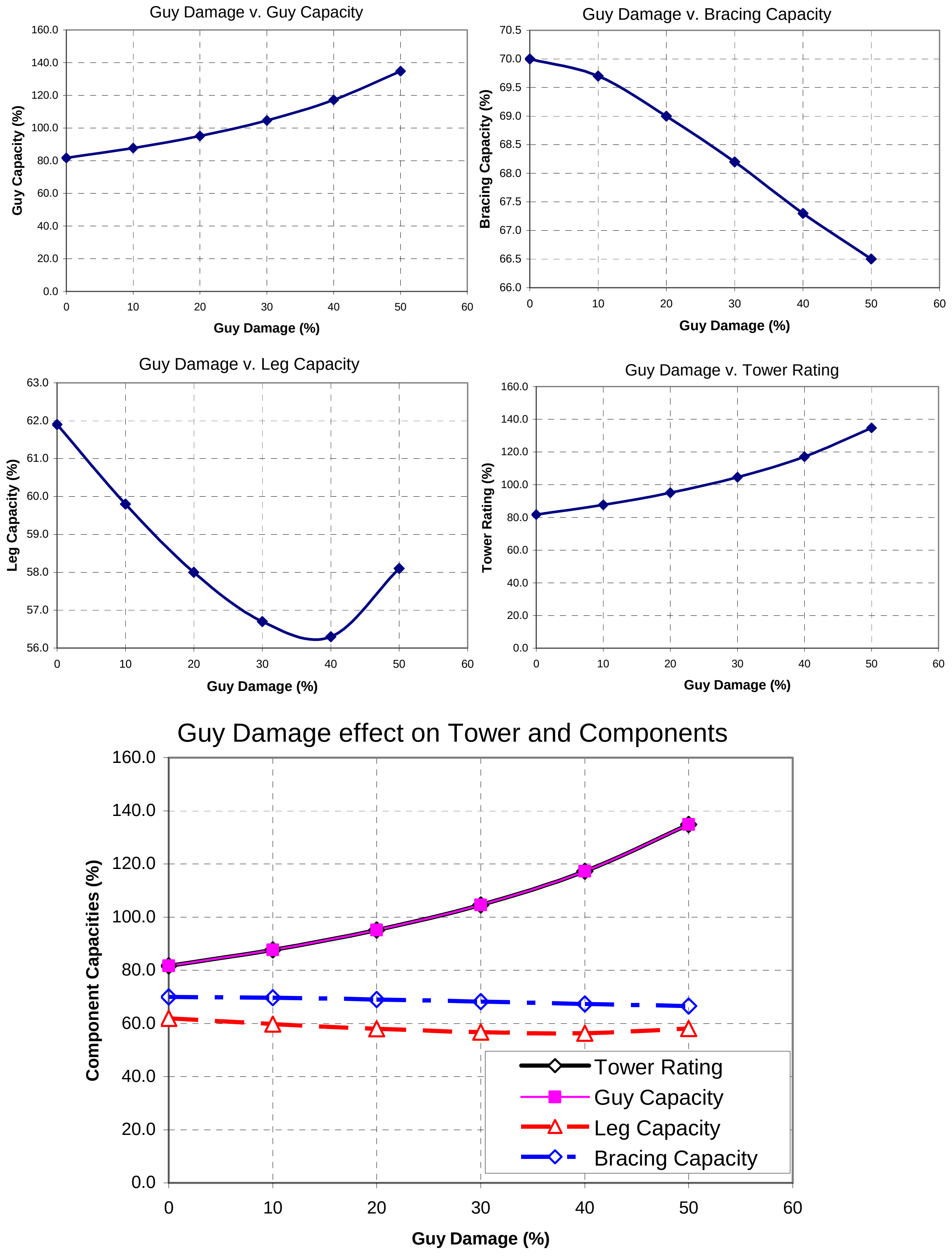


Figure 3.19. Effect of Guy Damage on Tower Rating and Components Capacities

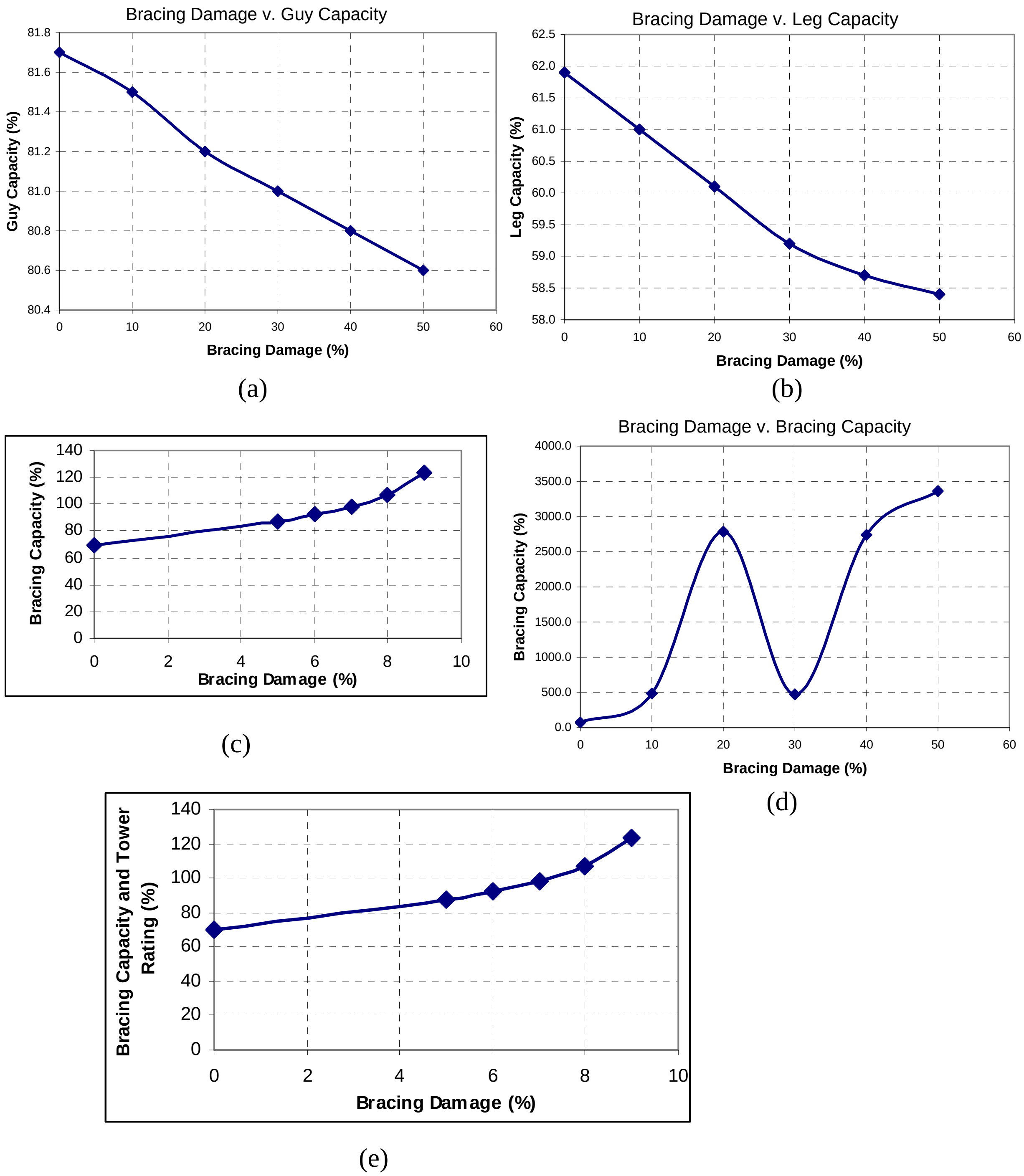


Figure 3.20. Effect of Braces Damage on Tower Rating and Components Capacities

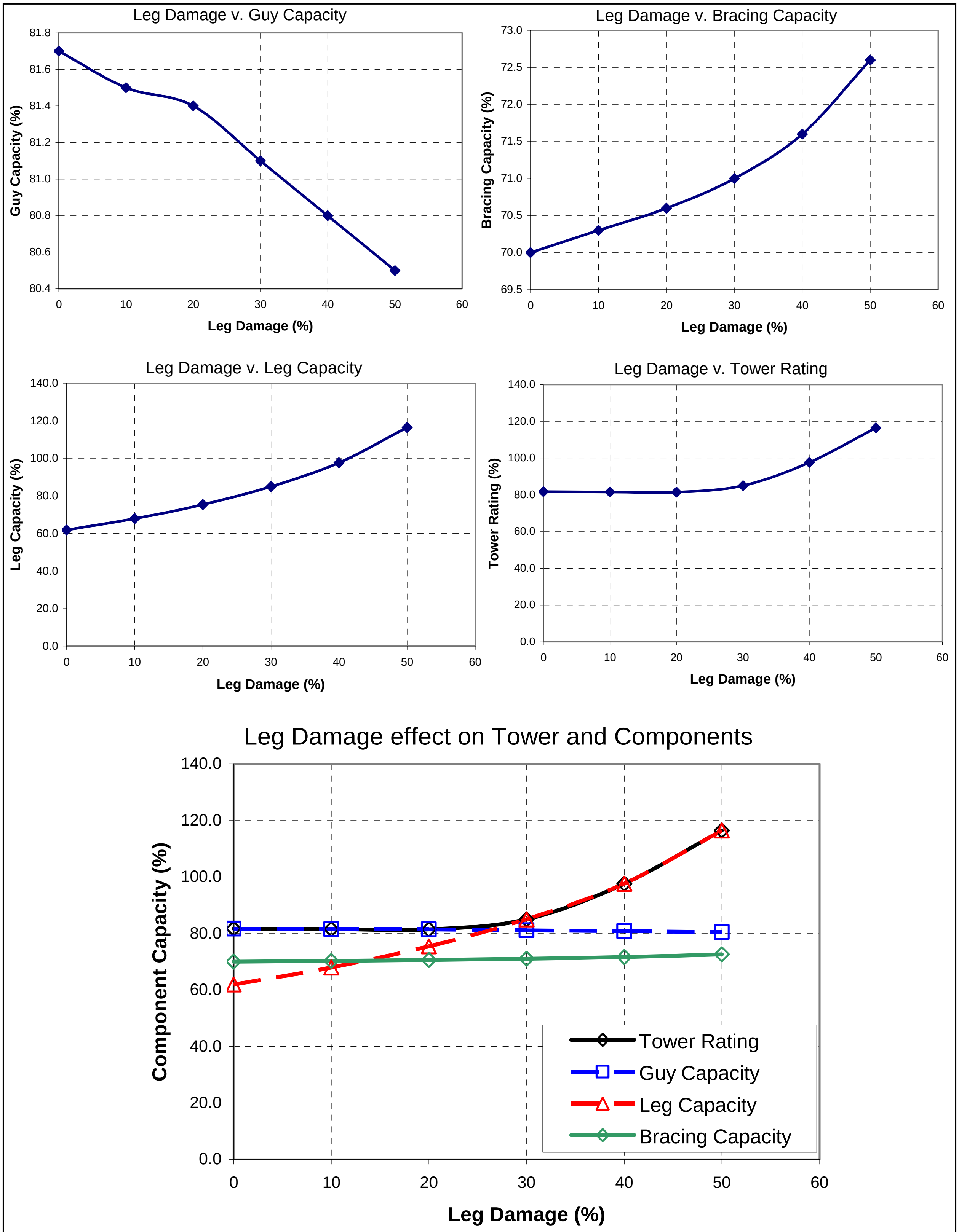


Figure 3.21. Effect of Leg Damage on Tower Rating and Components Capacities

The member forces in the critical braces identified earlier were investigated in the parametric study (Figure 3.22). 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 3.23. 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 3.24 through 3.27. General observations from the analyses include the following:

- Braces at the location of the guy-to-tower connections were stressed the most.

- Damage to the tower was introduced at a reduction in member cross sections. The forces in the critical braces decreased as the legs and braces were increased. On the other hand, the forces in the critical braces increased when the damage to the cables and the whole tower increased.

- The forces in the critical cables decreased as the damage to the guys and the whole tower was increased. Whereas the forces in the critical cable guys increased as the damage to the braces and legs increased.

- The forces in the critical tower legs decreased as the damage to the braces and legs increased, whereas the forces increased as the damage to the guy cables and overall tower increased.

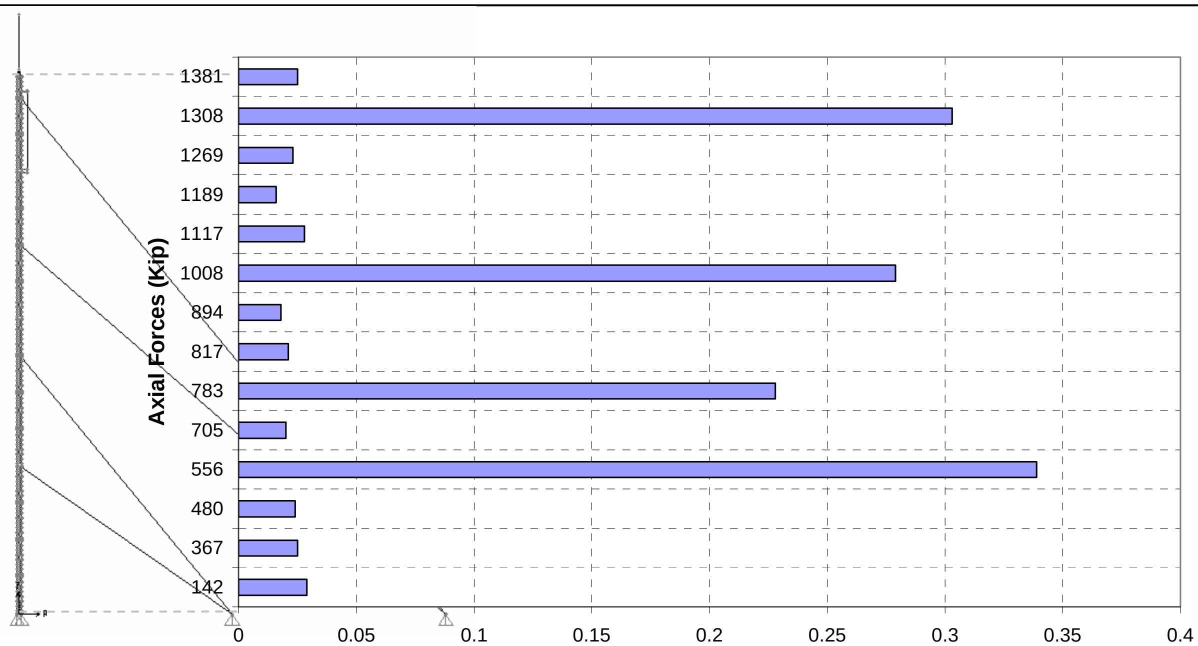


Figure 3.22. Axial Forces in Selected Critical Braces of the Tower

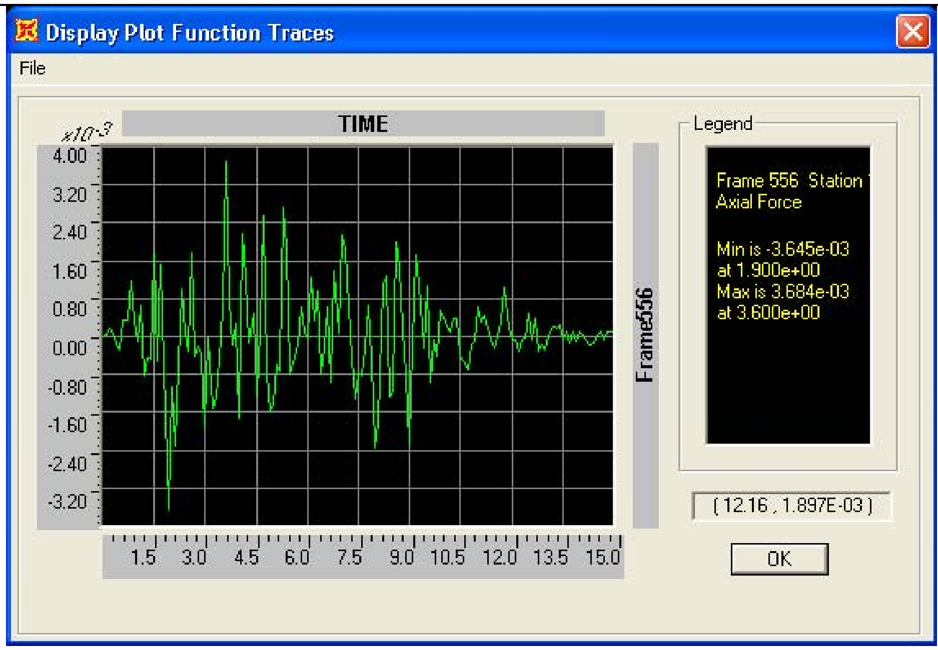


Figure 3.23. Axial Force in kips with Time at Selected Brace 556 for Time History

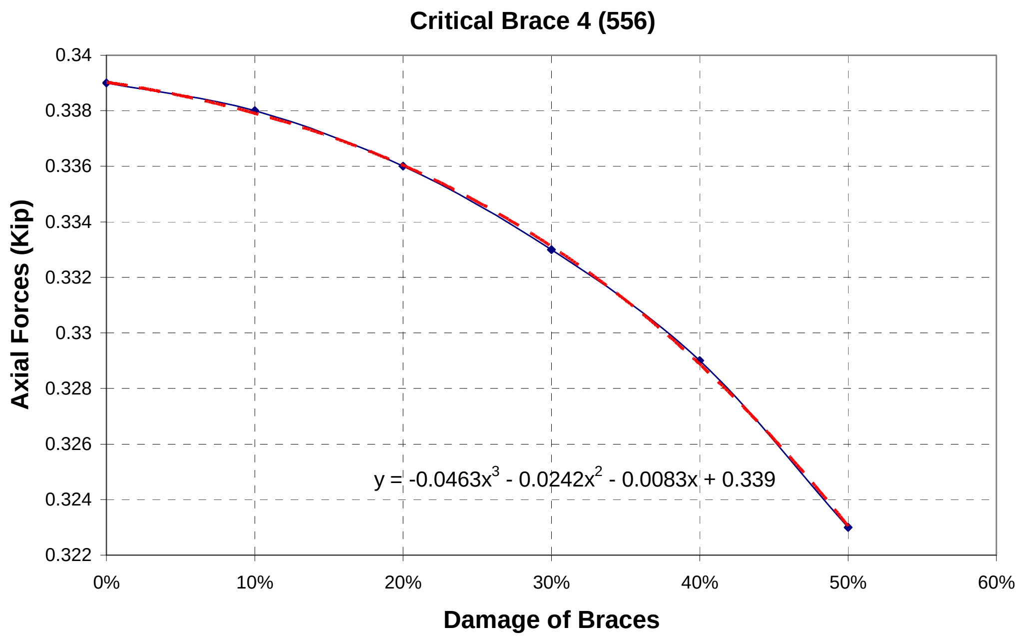


Figure 3.24. Axial Force for Critical Brace # 556 with Damage to Braces

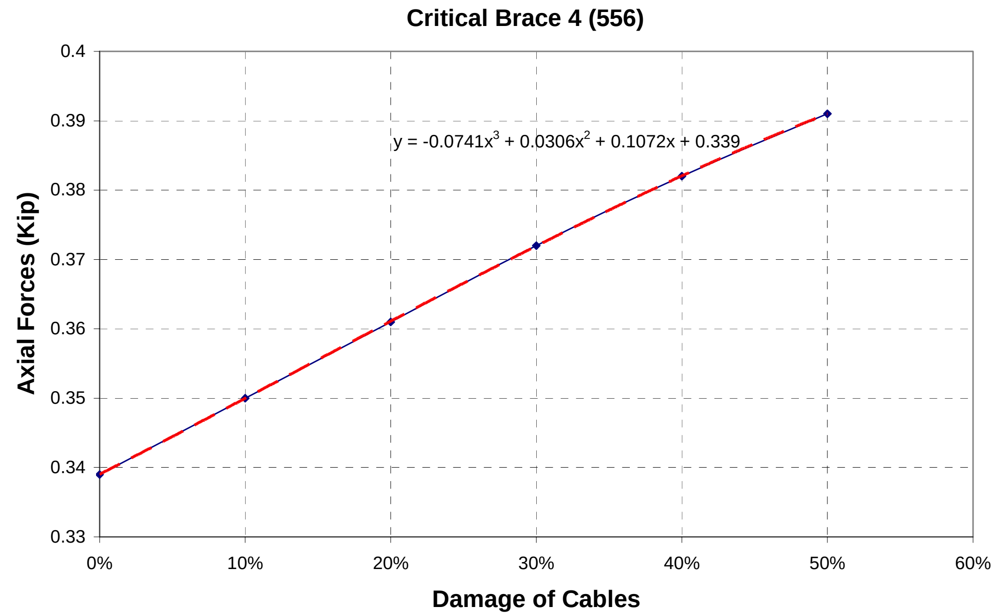


Figure 3.25. Axial Force for Critical Brace # 556 with Damage to Cables (Guys)

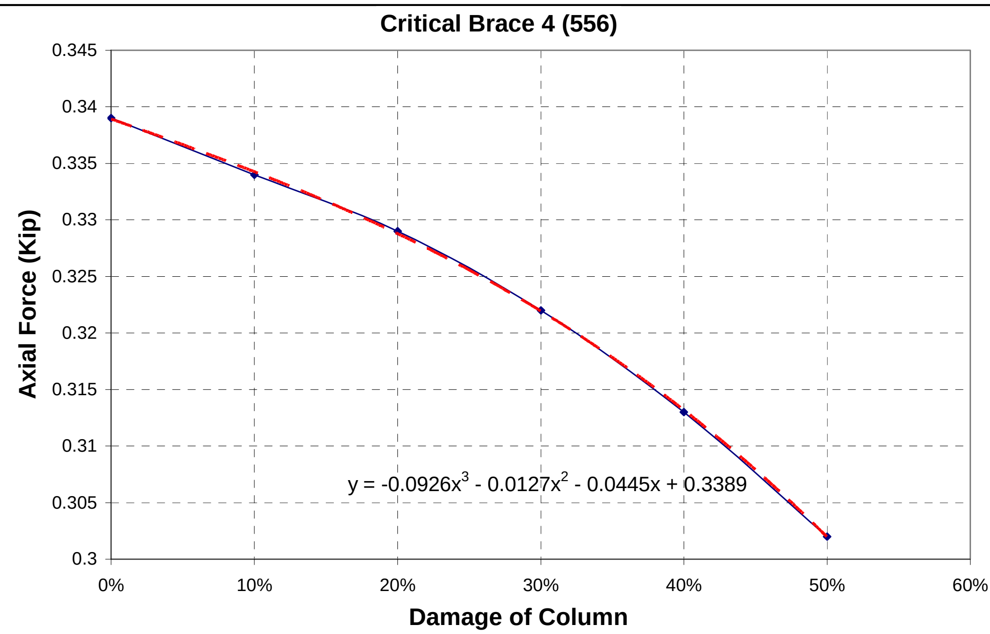


Figure 3.26. Axial Forces for Critical Brace # 556 with Damage to Legs

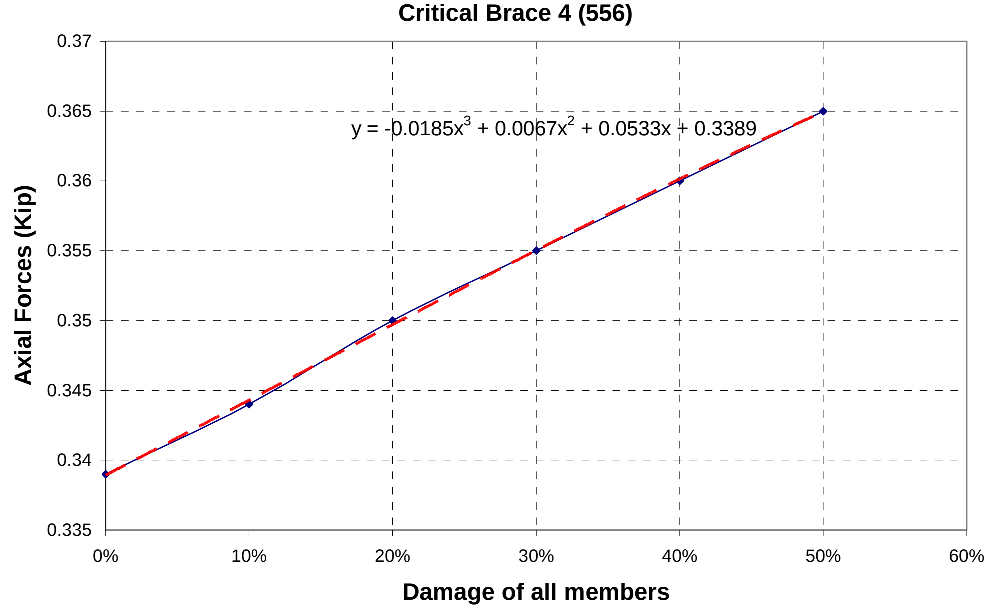


Figure 3.27. Critical Brace #556 with damage to all members (legs, guys, and braces)

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 3.28). 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 3.29. As mentioned previously, the guy cables were modeled in SAP using bar elements. The apparent compressive axial forces shown in Figure 3.29 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 3.30 through 3.33. 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.

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 3.34). 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 3.35. 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 3.36 through 3.39.

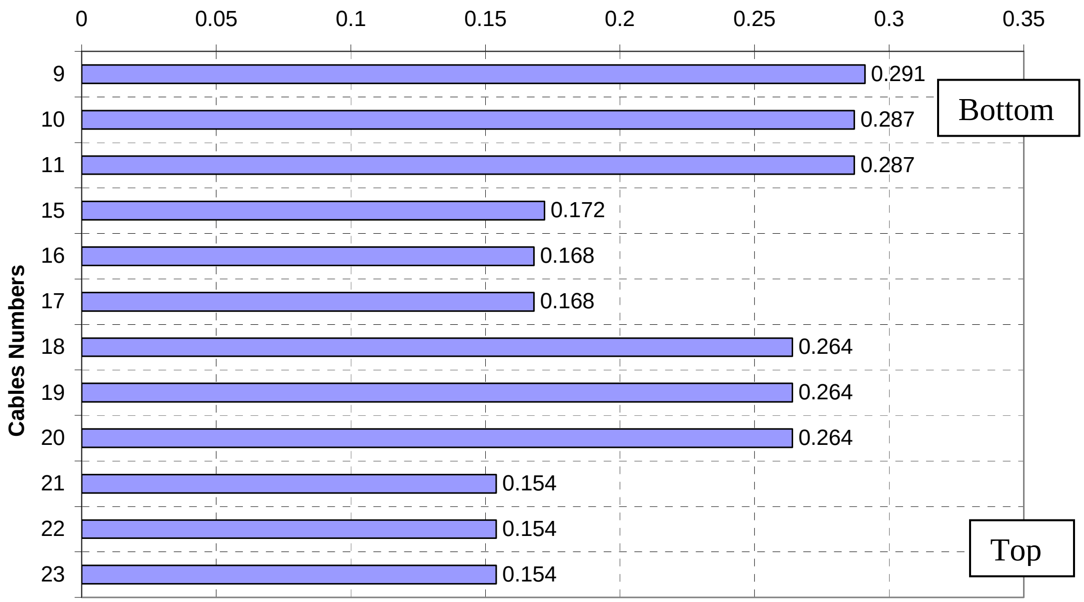


Figure 3.28. Axial Forces in Cables (kips) without any Damage in the Tower

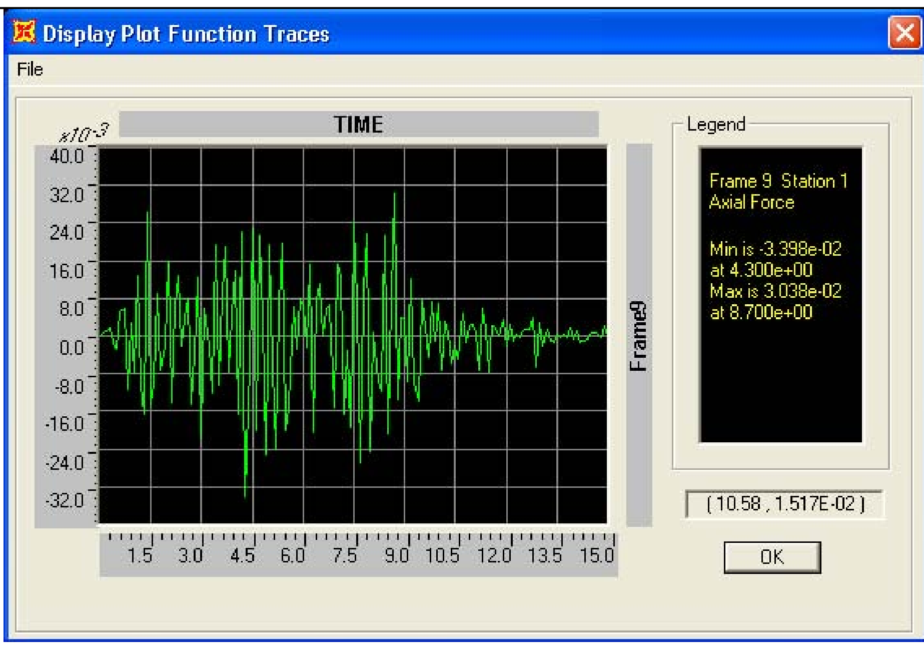


Figure 3.29. Axial Forces with Time at Selected Cable 9 for Time History Function

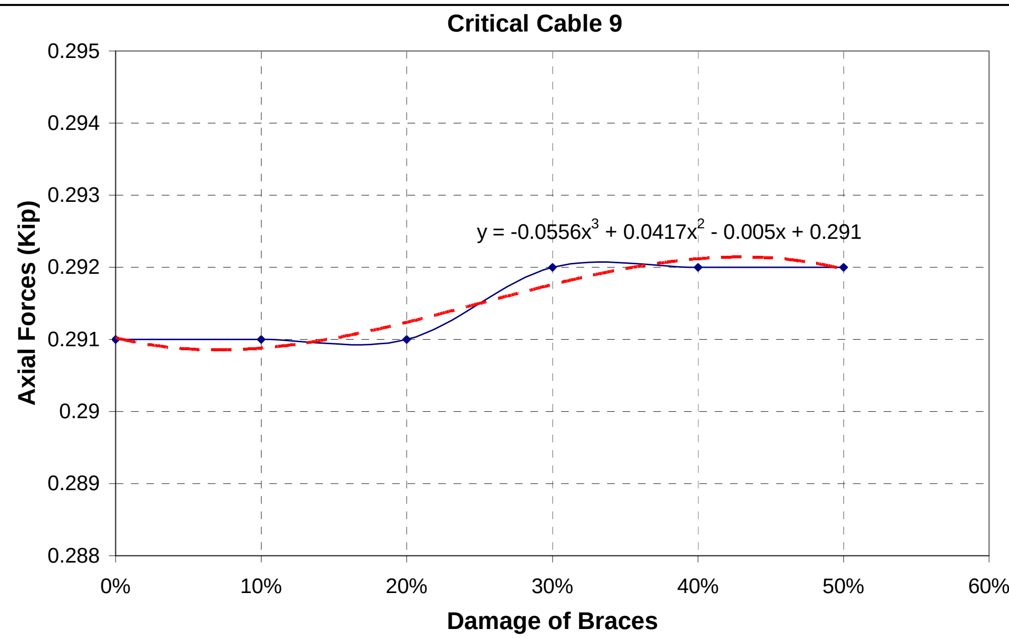


Figure 3.30. Variation of Axial Force in Cable 9 with Damage to Braces

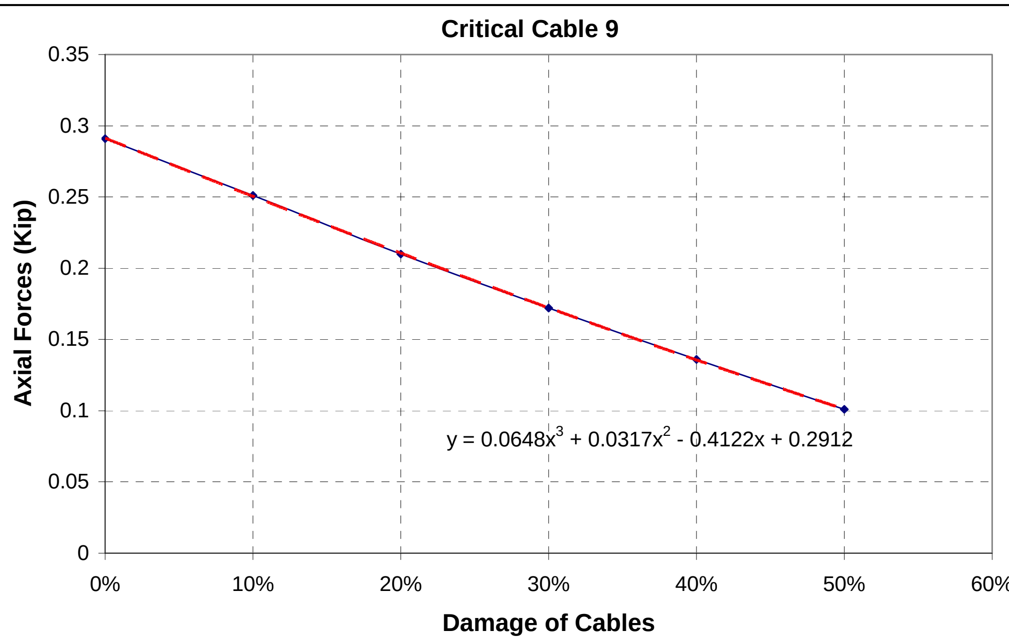


Figure 3.31. Variation of Axial Force in Cable 9 with Damage to Guy Cables

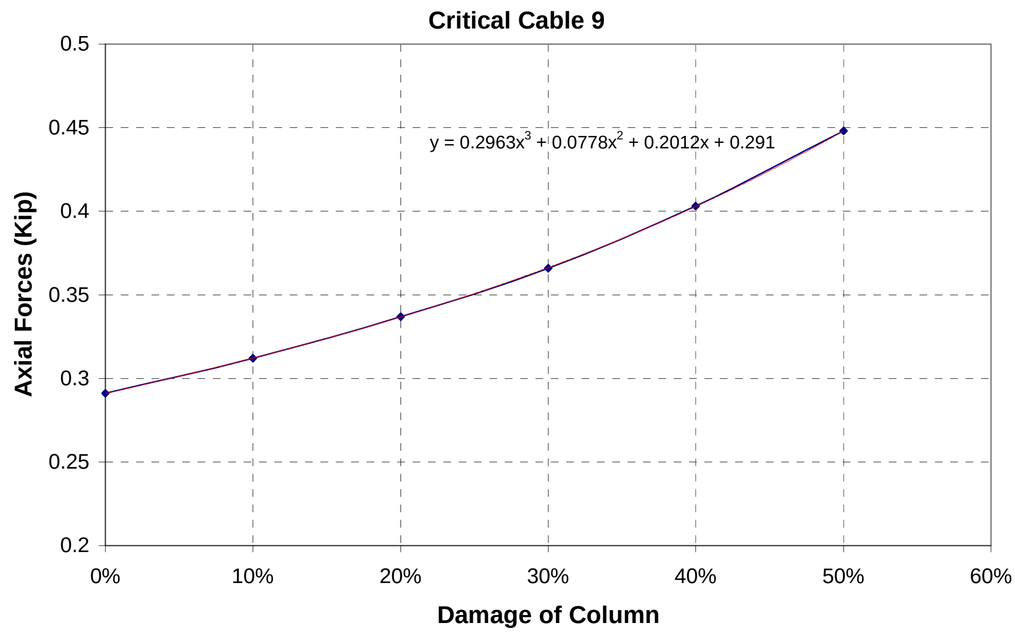


Figure 3.32. Variation of Axial Force in Cable 9 with Damage to Legs

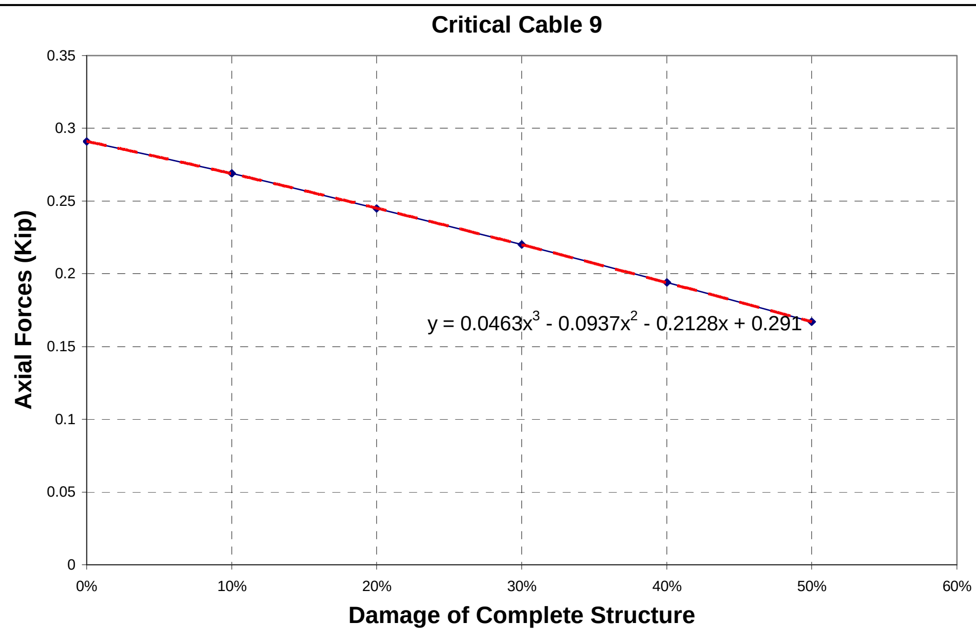


Figure 3.33. Variation of Axial Force in Cable 9 with damage to all members

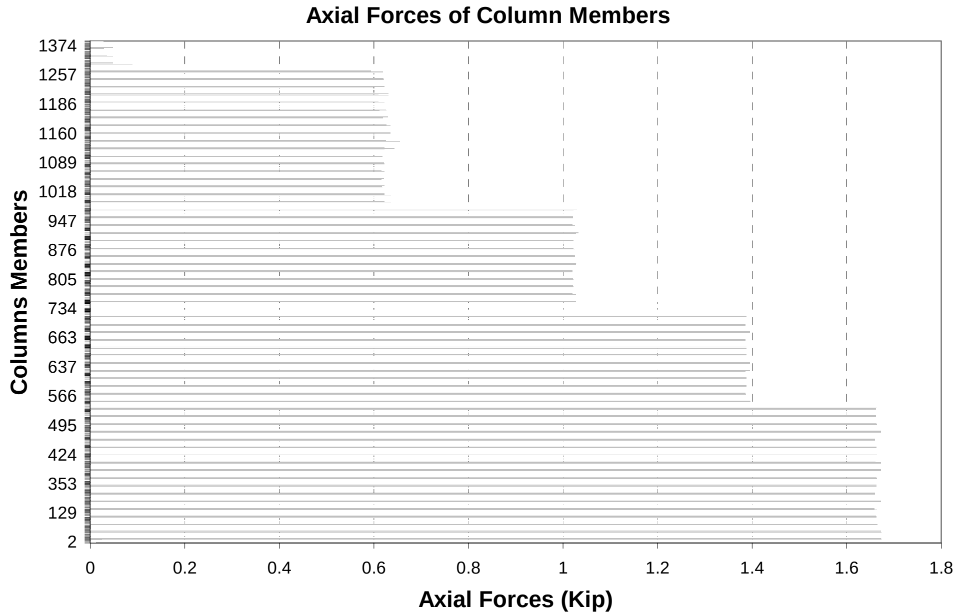


Figure 3.34. Axial Forces of All Leg (column) Members

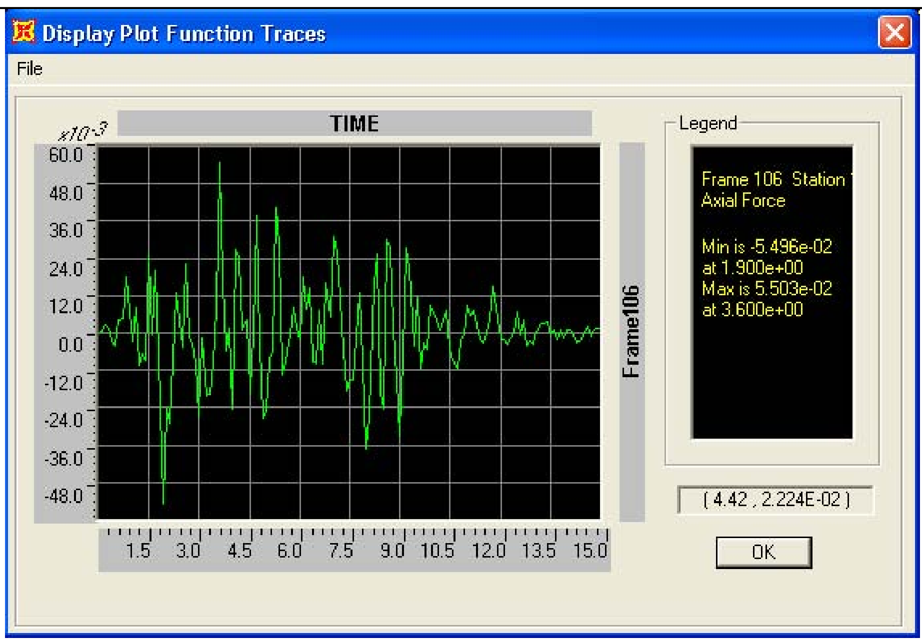


Figure 3.35. Axial Forces with Time for Leg Member # 106 for Time History Function

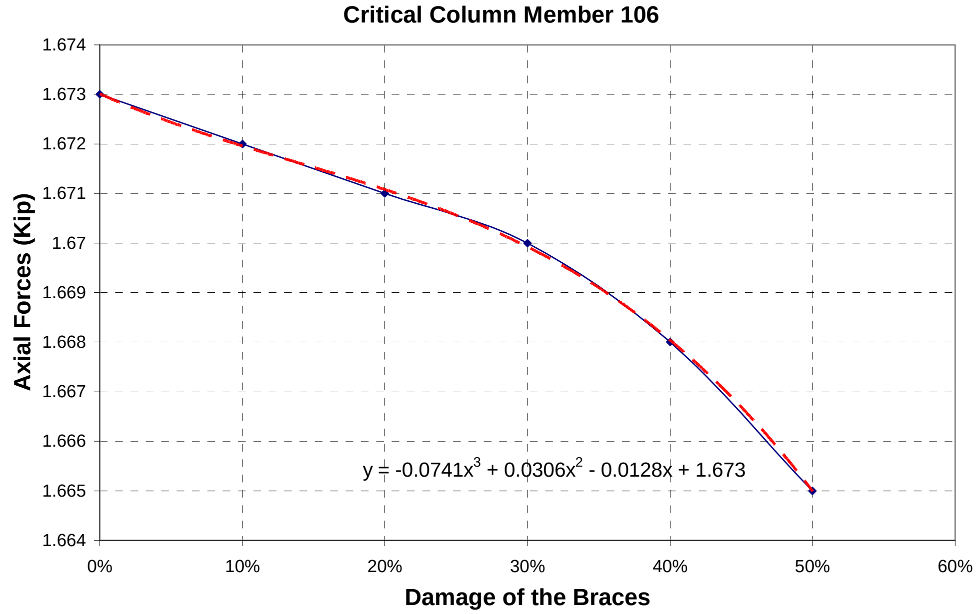


Figure 3.36. Axial Force in Leg Member # 106 with Damage to Braces

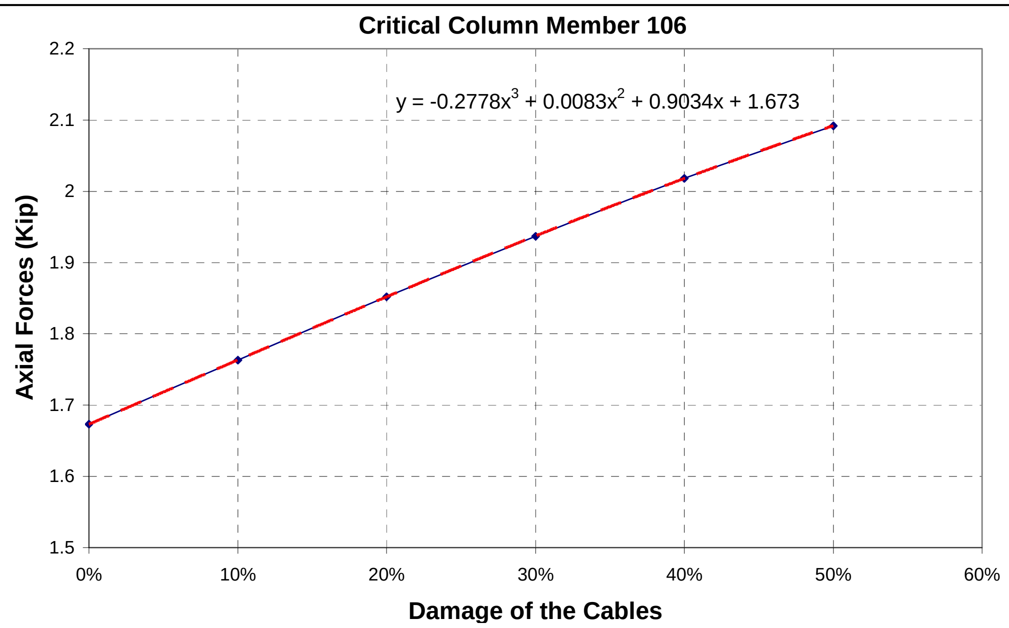


Figure 3.37. Axial Force in Leg Member #106 with Damage of Cables

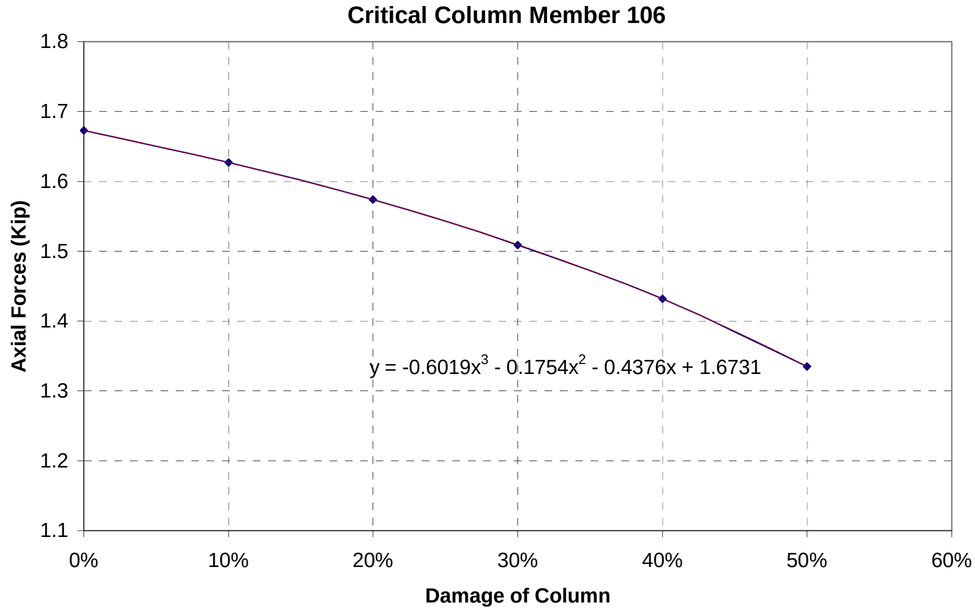


Figure 3.38. Axial Forces in Leg Member # 106 with Damage of Column

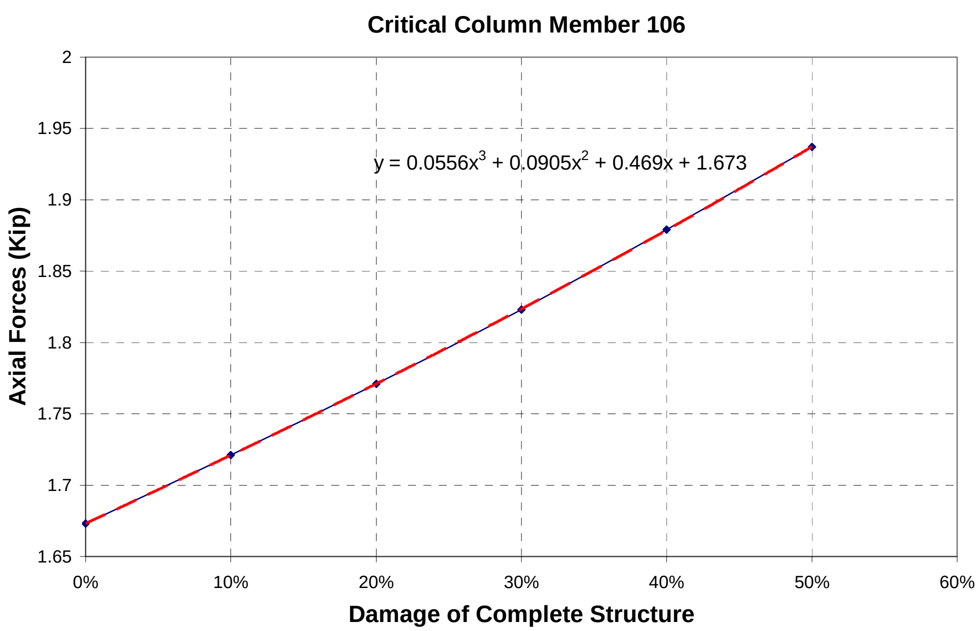


Figure 3.39. Variation of Axial Force in Leg #106 with damage to all members

3.5 Analysis of Kansas City Tower

3.5.1 Development of ERITower Model

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 in 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. Approximations were based on the relations of sizes of other members in the same areas.

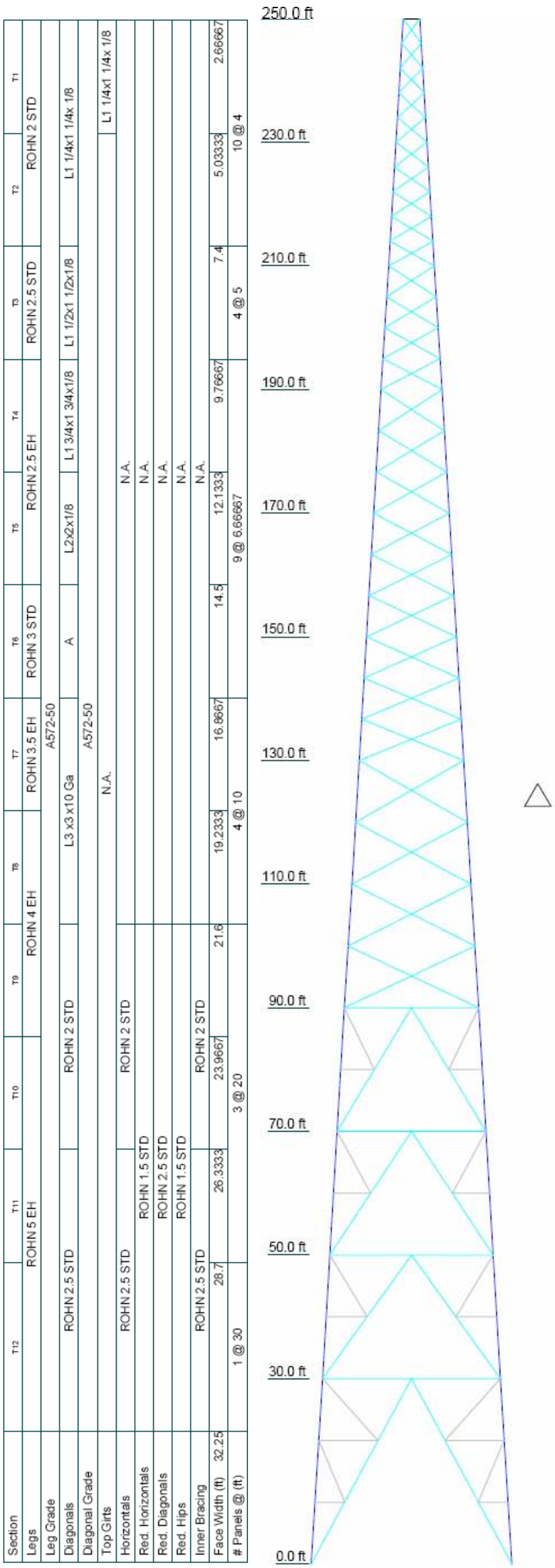
For this tower there were 12 separate sections. The sections were labeled T1 to T12 with T12 being 30 feet tall and the remaining sections being only 20 feet. Antennas and feedlines were placed on the tower based on information given by MoDOT. Two DB 224 antennas were placed on the tower on the same leg. The upper DB224 dipole antenna is mounted so that its very 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 3.40a and 3.40b, respectively. Results of the wind and ice analysis are summarized in the following.

3.5.2 Wind and Ice Analysis

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 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 .75 was used when ice is included. The load combinations are then found to be:

$D + 1.0(W_O)$: without ice
$D + .75(W_I) + I$: with ice

Figure 3.40. (a) Select Photographs of Kansas City Tower



APPURTENANCES

TYPE	ELEVATION	TYPE	ELEVATION
DB224	249 - 228	DB224	225.5 - 204.5

SYMBOL LIST

MARK	SIZE	MARK	SIZE
A	L2 1/2x2 1/2x 10 Ga		

MATERIAL STRENGTH

GRADE	Fy	Fu	GRADE	Fy	Fu
A572-50	50 ksi	65 ksi			

TOWER DESIGN NOTES

1. Tower designed for Zone A - 30 psf/22.5 psf w/0.50 in ice to the EIA-222-C Standard.
2. Wind pressure multiplier is 0.75 for the ice condition.

Figure 3.40. (b) ERITower Model
of the Kansas City Tower

Using Standard TIA-222-F: 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. 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 (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, 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.

3.5.3 Key Results

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.

$$\frac{Combined \quad Stress \quad Ratio}{Allowable \quad Stress \quad Ratio} \leq 1.0$$

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 3.12 and 3.13 show the critical components in the tower and their capacities for the TIA-222-C and TIA-222-F standards, respectively.

It can be seen from Table 3.12 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 3.13 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 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 3.14.

Table 3.12. 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 3.13. 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 3.14. 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

3.6 Modeling Summary and Conclusions

Modeling described in this chapter included the analysis of the Taum Sauk tower using SAP and ERI Tower software. The SAP model was used for seismic analysis whereas the ERI Tower software was used for wind and ice loading analysis and design using ANSI/TIA/EIA-222-F code. The KC tower was analyzed only under wind and ice loads using ERI Tower software using ANSI/TIA/EIA-222-C and -F code versions. Below are the main observations, and corresponding recommendations.

3.6.1 Dynamic Modeling of Taum Sauk Tower

- The results of the SAP dynamic modeling indicates that the tower members are not near their capacities under the standard spectrum loading function.

- In general, the braces at the location of the guy-to-tower connections were stressed the most.

- Damage to the tower was simulated as a reduction in member cross section (e.g., resulting from corrosion). Forces in the critical braces decreased as the legs and braces were increased. On the other hand, the forces in the critical braces increased when the damage to the guy cables and the tower as a whole increased.

- Forces in the critical cables decreased as the damage to the guys and the tower as a whole was increased. Forces in the critical cable guys increased as the damage to the braces and legs increased.

- Forces in the critical tower legs decreased as the damage to the braces and legs increased, whereas the forces increased as the damage to the guy cables and overall tower increased.

3.6.2 Wind and Ice Modeling of Taum Sauk Tower

- The tower passes with respect to wind and ice loading according to TIA-222-F code specifications. The stresses in the tower members in terms of their capacities ranged from 4.9% in the horizontal brace to 81.7% in the top guy.

- The diagonal braces are the most stressed members in the tower structure at a 70% of their capacities.

- Damage to the guy cables only causes the tower to fail at about 25% reduction in 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 causes the tower rating to exceed its capacity. This indicates that the braces can result in tower instability and collapse with very little damage.

- Damage to the tower legs as high as 42% will cause 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, however, that tower legs can corrode faster if moisture gets trapped inside the tower legs.

3.6.3 Wind and Ice Modeling of Kansas City Tower

- Using the TIA-222-C code, the tower is loaded to 101.7% capacity. The capacity was controlled by the diagonal angles at 90 to 110 feet elevation. Generally, the remaining member capacities were below 50% utilized. This indicates that these diagonal members were most critical of the tower.

- Using the TIA-222-F code with an allowable stress ratio of 1.0, the tower fails. Failure is controlled by the diagonal angles.

- Since the F-code permits for the allowable stress ratio to be as high as 1.333, the analysis indicates that the tower will pass with a rating of 88.9% of available capacity. The critical member is still the diagonal, but at a higher elevation of 90 to 110 ft.

3.6.4 Recommendations

- The models for the towers were based on available drawings. Some field measurements were collected to verify the drawings to fill-in the missing data. It is recommended that additional field measurements be collected to exactly represent the physical conditions as they exist in the field.

- All attachments shown on the drawings were included in the models. The existing towers have additional attachments that were not incorporated in the analytical models. It is recommended that the analyses be performed with all attachments as per the field conditions.

- It was apparent from the parametric study that damage to tower components can cause severe consequences and failure, thus it is recommended that the existing tower conditions be identified through detailed field inspection and condition indexing (e.g., Chapter 2). Nondestructive techniques may be considered to evaluate internal member integrity.

- For guyed towers, the condition of the cables and the tension levels can affect the response of the tower to earthquake, wind, and ice loading. This information was not available for the tower analyzed and it is recommended that future work include such conditions to accurately represent the tower condition in the field.

4. Database Development

A centralized electronic database and inventory has been developed to organize the radio tower network via an internal (MoDOT server) site on the World Wide Web (WWW). Informational items that may be included in the database include the following:

Site name
Tower location (county, latitude and longitude)
Tower height and model
Current condition index (developed following the procedures described in Chapter 2)
Date constructed
Structural drawings
Tower Attachments and appurtenances (*i.e.*, antennae, dishes, hardware, etc.)
Digital photos of tower and appurtenances (if available)
Foundation and soil type
Links to electronic copies of any available inspection reports or design specifications

Dreamscape software was used to develop the web page system. Figure 4.1 shows a “screen shot” of the main (opening) page of the database, which is the tower location map presented previously in this report as Figure 1.1. The main page also contains links to seismic, wind, and ice loading maps for the state of Missouri. Critical tower locations are superimposed within the seismic, wind, and ice contours (see Figure 4.2).

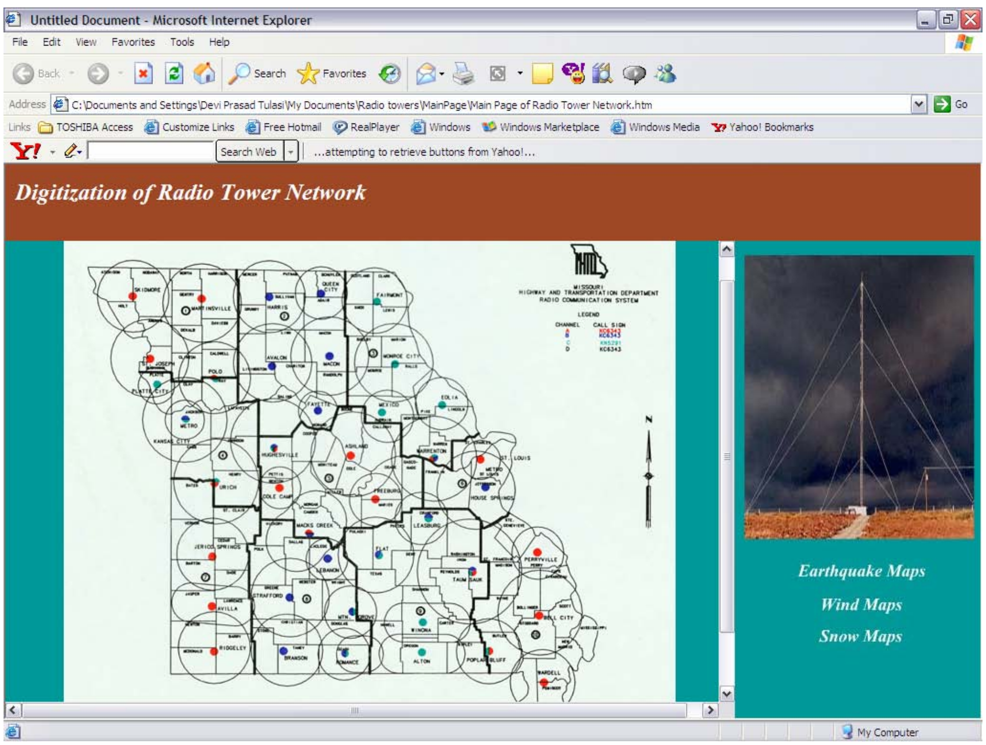
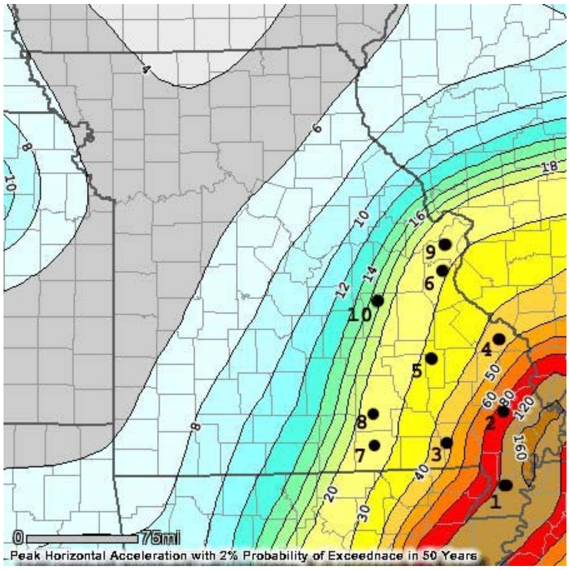
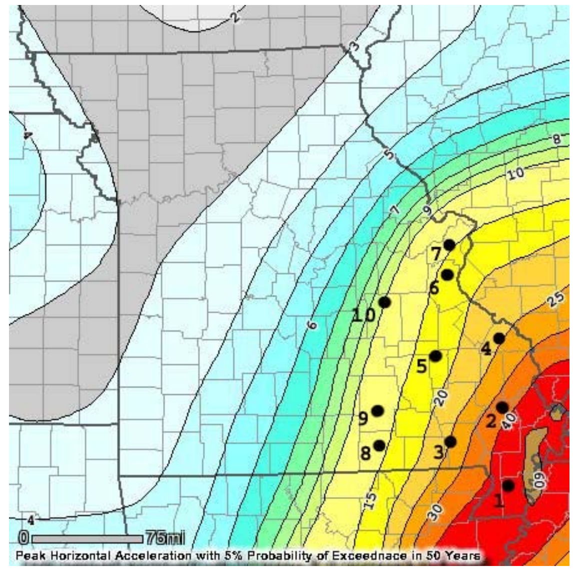


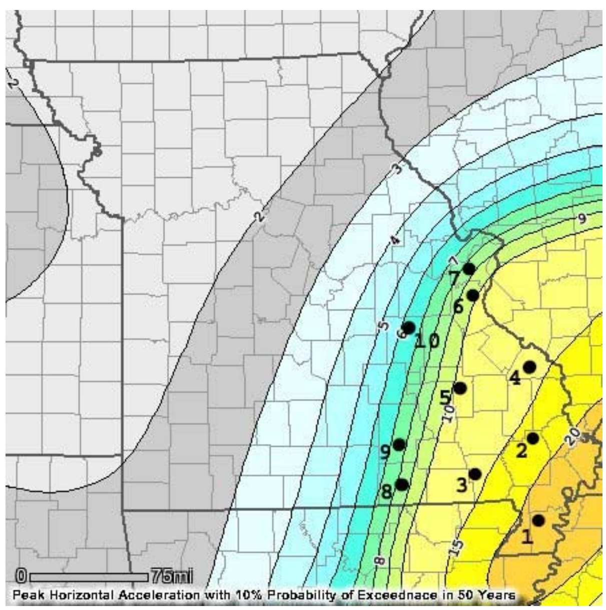
Figure 4.1. Main web page of the electronic database.



(a)



(b)



(c)

Figure 4.2. Sub-screens showing peak horizontal seismic acceleration (USGS) contours and relative locations of critical towers: (a) 2% PE in 50 years, (b) 5% PE in 50 years, (c) 10% PE in 50 years.

Sub-pages containing detailed information for each tower may be accessed by clicking on the tower locations (small colored circles) on the main page. Figure 4.3, for example, shows the web page for the Skidmore Tower. Digitized structural drawings and specifications for that particular tower may also be accessed from this page (e.g., Figure 4.4). The condition index (CI) for the tower may be calculated or updated after a new inspection by clicking on the "CI" button, which opens the page shown as Figure 4.5.

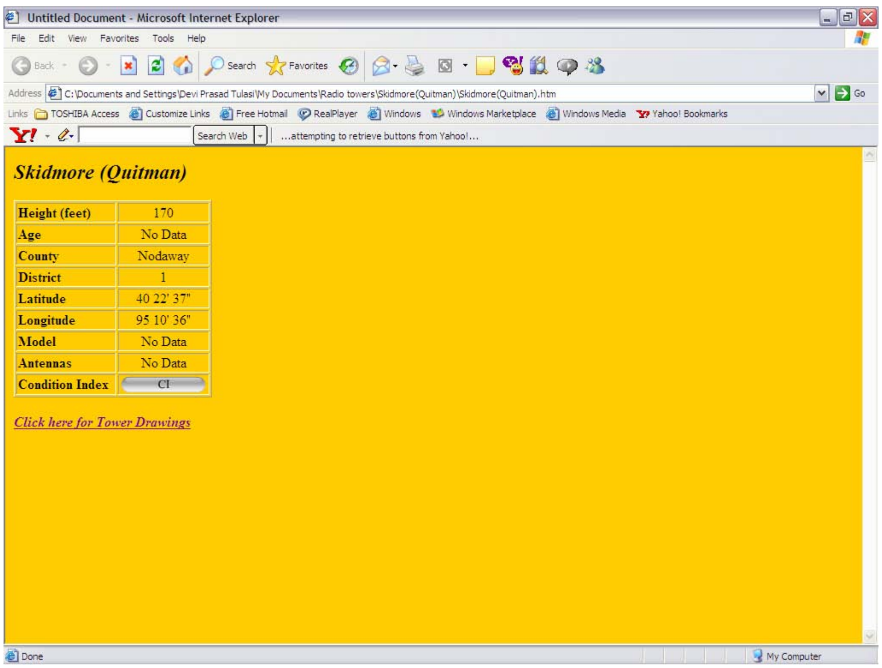


Figure 4.3 Example of a sub-screen showing tower information and links to condition indexing system and structural drawings/specifications.

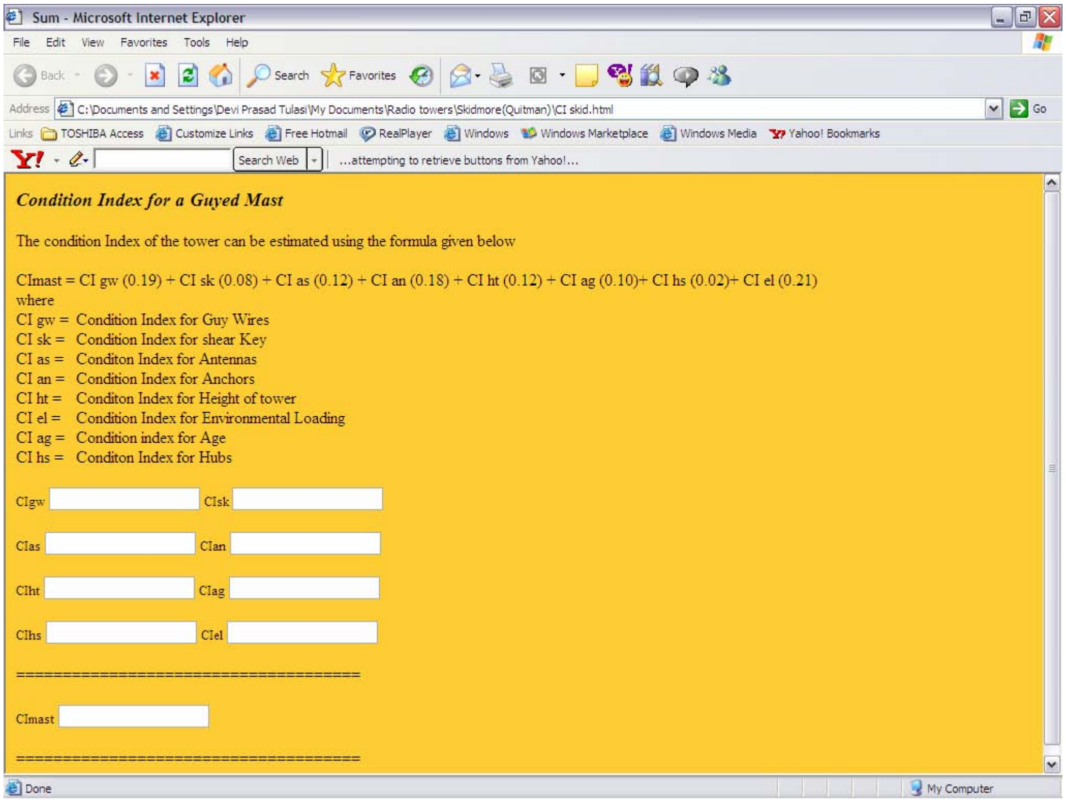


Figure 4.4. Example of sub-screen for calculating or updating condition index.

5. Summary, Conclusions, and Recommendations

5.1 Summary and Key Conclusions

Current and future reliance on the MoDOT two-way communications system has generated significant motivation to assess the physical condition of the radio tower network that supports it. Many of the towers are over 40 years old and in relatively poor physical condition. The following major tasks have been undertaken and documented in this report:

- Reviewed existing methodologies for function-based condition assessment of civil infrastructure. Reviewed historical failure mechanisms in free standing and guyed communications towers. Identified and reviewed appropriate design standards and dynamic analysis procedures.

- Developed a rational “condition indexing” (CI) system that may be used to systematically quantify the physical condition of individual towers in the network. Output from the indexing system is a number ranging from 0 to 100 that may be used to prioritize mitigation and repair operations and to allocate resources accordingly.

- Demonstrated use of the condition indexing system for towers located at Taum Sauk (Iron County) and Ashland (Boone County).

- Conducted detailed structural analysis of the Taum Sauk tower under seismic, wind and ice loading and the Kansas City tower under wind and ice loading. Modeling included analysis of the Taum Sauk tower using SAP and ERI Tower software. The SAP model was used for seismic analysis whereas the ERI Tower software was used for wind and ice loading analysis and design checks using ANSI/TIA/EIA-222-F code. The KC tower was analyzed under wind and ice loads using ERI Tower software using ANSI/TIA/EIA-222-C and -F code versions.

- Conducted a parametric modeling analysis to evaluate the effects of simulated deterioration (e.g., corrosion) in the guy cables, braces, and axial members of the Taum Sauk tower.

- Digitized relevant information regarding the tower network and the individual towers (e.g., location, appurtenances, structural drawings, condition index, etc.) in the form of a centralized, web-based electronic format. An electronic web-based database was developed for implementation into management of the tower network. The data base includes an interactive map of Missouri where the user may click on a select tower location to access physical specifications of the select tower (e.g., type, height, etc), scanned copies of the original tower structural drawings, and an interactive screen for entering and updating the current condition index (CI) of the tower. The database may be uploaded to an FTP site for internal MoDOT access.

Key conclusions and findings from the research include the following:

1) A systemic condition indexing (CI) procedure was developed to assess the physical condition of guyed communication towers using the following equation:

$$CI_{gt} = CI_{gc}(0.31) + CI_{ga}(0.19) + CI_{cm}(0.31) + CI_{fd}(0.19)$$

where CI_{gt} is the overall condition index, CI_{gc} is the component condition index of the guy cables, CI_{ga} is the component condition index of the guy anchors, CI_{cm} is the component condition index of the central mast, and CI_{fd} is the component condition index of the foundation. Component condition indices are assigned based on observed deviations from the ideal conditions defined in Table 2.8. The weighting factors were developed by considering cause and effect interactions among the primary tower components and environmental loads from wind, ice, precipitation, and earthquakes. The overall condition index for the tower ($0 < CI < 100$), may be correlated to a qualitative description and recommended action as summarized in Table 2.11.

2) The overall CI for the Taum Sauk tower is 54 out of 100, which, following Table 2.11 corresponds to "Fair: Moderate deterioration but function is still adequate." Economic analysis of repair alternatives is recommended. This analysis may be guided by considering the corresponding effects to condition index under different repair strategies. If, for example, the guy cables are replaced such that CI_{gc} increases to 100, the overall CI for the tower will increase to 70.

3) The overall CI for the Ashland tower is 85 out of 100, which corresponds to "Excellent: No noticeable defects; some aging or wear may be visible." No immediate action is warranted.

4) Weighting factors are suggested to adjust CI for subjective factors such as the importance of the tower to the overall functioning of the state-wide. A series of factors based on the number of adjacent towers served by a candidate tower is suggested.

5) The primary threats of tower damage or structural failure come form three sources: excessive wind loading, ice loading, and seismic loading. Search of the literature does not describe in detail any specific cases of tower damage or failure due to earthquake loading in the U.S. The majority of historical failures as the result of dynamic loading have been associated with wind and ice loading effects.

6) Results of the SAP dynamic modeling indicates that the Taum Sauk tower members are not near their capacities under the standard spectrum seismic loading function. In general, the braces at the location of the guy-to-tower connections were stressed the most (70% of total capacity).

7) The Taum Sauk tower passes with respect to wind and ice loading according to TIA-222-F code specifications. The stresses in the tower members in terms of their capacities ranged from 4.9% in the horizontal brace to 81.7% in the top guy.

8) Damage to the Taum Sauk tower was simulated by reducing the cross sectional areas of the guy cables, diagonal braces, and axial leg members (e.g., resulting from corrosion). Forces in the critical braces decreased as the legs and braces were increased. Forces in the critical braces increased when the damage to the guy cables and the tower as a whole increased. Damage to the guy cables causes the tower to fail at about 25% reduction in guy cross section. Damage to the guys relieves the loads on the bracings and legs. Little damage to the braces causes the tower rating to exceed its capacity. This indicates that the braces can result in tower instability and collapse with very little damage. Damage to the tower legs as high as 42% will cause 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, however, that tower legs can corrode faster if moisture gets trapped inside the tower legs.

9) Using the TIA-222-C code, the Kansas City tower is loaded to 101.7% of available capacity. The capacity was controlled by the diagonal angles at 90 to 110 feet elevation. Generally, the remaining member capacities were below 50% utilized. This indicates that these diagonal members were most critical of the tower.

10) Using the TIA-222-F code with an allowable stress ratio of 1.0, the Kansas City tower fails. Failure is controlled by the diagonal angles. However, since the F-code permits the allowable stress ratio to be as high as 1.333, analysis indicates that the tower will pass with a rating of 88.9% of available capacity. The critical member is still the diagonal, but at a lower elevation of 90 to 110 ft.

5.2 Recommendations

1) The proposed conditioning indexing system should be expanded and modified for applicability to free-standing and monopole towers.

2) The CI system should be used to rank guyed towers in the MoDOT network. Once an initial CI is assigned to each tower, subsequent inspections and maintenance should be performed on a schedule as follows (from TAI/EIA 222-G):

    a) at a minimum of three-year intervals for guyed masts and five-year intervals for self-supporting structures.
    b) After severe wind, ice, or earthquake loadings
    c) Shorter inspection intervals are required for structures in corrosive atmospheres or subject to frequent vandalism.
    d) After a change in type, size, or number of appurtenances such as antennas, transmission lines, platforms, ladders, etc.
    e) After any structural modifications

f) After any change in serviceability requirements or land use surrounding the structure

3) Decisions regarding repair to towers identified as deficient should be made in consultation with and external Tower Design, Analysis and Maintenance Consultant (e.g., Appendix F).

4) Site specific liquefaction analyses should be conducted for towers located in seismic zones 2A and 3 (see Figure 3.5).

5) Populate and continuously update the electronic web data base. Disseminate web site to regional tower maintenance personnel.

6) Conduct more detailed dynamic modeling to address the following issues: (a) The models for the towers were based on the available drawings. Some field measurements were collected to verify the drawings to fill-in the missing data. It is recommended that additional field measurements be collected exactly represent the towers as they exist in the field; (b) All attachments shown on available drawings were included in the models. The existing towers have additional attachments that were not incorporated in the models. It is recommended that the analyses be performed with all attachments as per the field conditions; (c) incorporate measured guy tensions into dynamic models.

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Appendix A. Tabulated Tower Location Data

				Lat			Long		
District	Location	County	Height	Degees	Min	Sec	Degees	Minutes	Seconds
9	ALTON	OREGON	165	36	41	49	91	30	3
5	ASHLAND	BOONE	320	38	46	11	92	17	38
2	AVALON	LIVINGSTON	170	39	39	20	93	20	37
7	AVILLA	JASPER	270	37	11	35	94	8	32
10	BELL CITY	STODDARD	285	36	54	21	89	52	20
8	BRANSON	TANEY	170	36	44	46	93	16	36
7	CARTHAGE	JASPER	350	37	5	52	94	18	35
5	COLE CAMP	BENTON	70	38	27	43	93	13	8
3	EOLIA	PIKE	120	39	14	40	91	1	28
3	FAIRMONT	CLARK	130	40	19	56	91	55	14
2	FAYETTE	HOWARD	80	39	12	17	92	43	32
9	FLAT	PHELPS	320	37	45	22	91	58	28
5	FREEBURG	MARIES	130	38	16	12	91	56	30
3	HANIBAL	MARION	145	39	41	44	91	24	6
2	HARRIS	SULLIVAN	170	40	20	35	93	18	47
6	HOUSE SPRINGS	JEFFERSON	270	38	24	47	90	31	23
5	HUGHESVILLE	PETTIS	320	38	51	26	93	13	6
5	JEFFERSON CITY	COLE	172	38	34	40	92	11	49
7	JERICO SPRINGS	CEDAR	305	37	36	42	94	0	3
7	JOPLIN	JASPER	165	37	3	31	94	27	43
4	KANSAS CITY	JACKSON	270	39	3	18	94	28	58
9	LEASBURG	CRAWFORD	120	38	7	56	91	17	42
8	LEBANON	LACLEDE	70	37	40	19	92	39	45
5	MACKS CREEK	CAMDEN	320	37	58	30	92	53	36
2	MACON	MACON	225	39	44	20	92	27	30
1	MARTINSVILLE	HARRISON	170	40	20	2	94	9	31
3	MEXICO	AUDRAIN	100	39	9	40	91	49	46
3	MONROE CITY	RALLS	265	39	41	4	91	38	36
8	MOUNTAIN GROVE	WRIGHT	130	37	5	55	92	16	12
10	PERRYVILLE.	PERRY	80	37	43	14	89	53	55
4	PLATTE CITY	PLATTE	160	39	25	52	94	47	26
4	POLO	RAY	170	39	31	28	93	59	38
10	POPLAR BLUFF	BUTLER	220	36	48	31	90	28	17
2	QUEEN CITY	SCHYULER	100	40	25	1	92	33	47
7	RIDGELEY	BARRY	170	36	42	3	94	1	24
8	ROMANCE	OZARK	130	36	43	27	92	27	28
10	SIKESTON	SCOTT	100	36	52	49	89	34	52
1	SKIDMORE (QUITMAN)	NODAWAY	170	40	22	37	95	10	36
8	SPRINGFIELD	GREENE	350	37	14	24	93	13	52
8	STRAFFORD	GREENE	285	39	19	18	93	7	46
1	ST. JOSEPH	BUCNANAN	220	39	48	8	94	48	53
9	TAUM SAUK (ARCADIA)	IRON	170	37	34	3	90	43	23
6	TOWN AND COUNTRY	ST. LOUIS	145	38	38	24	90	31	0
4	URICH	HENRY	320	38	23	25	94	0	44
10	WARDELL	PEMISCOT	170	36	20	59	89	48	22
3	WARRENTON	WARREN	220	38	44	7	91	12	52
9	WILLOW SPRINGS	HOWELL	220	37	0	36	91	59	2
9	WINONA	SHANNON	185	37	2	18	91	19	21

Appendix B: Detailed Description of CI Development Procedures – Earth Dam Example

Andersen and Torrey (1995) describe several steps required to develop a "function-based" conditioning indexing system. The CI system described in this report for communication towers is based on the following simplified synthesis of those steps. Each is described in more detail in the following sections within the context of the Andersen and Torrey (1995) CI system proposed for USACE earth dams.

4) Identify the functional components of the system.

5) Develop a component interaction matrix.

6) Code the interaction matrix to represent the strength of each interaction.

7) Define ranges between ideal and failed conditions for each component.

8) Develop weighting factors and formulate condition index scalar.

Step 1: Identify the functional components of the system

Most civil structures are complex systems comprised of numerous closely related and highly interactive functional components. Each of these components contributes in a different way to meet the overall objective of the structure. The overall objective of an earth dam, for example, is to retain a body of water or reservoir for an extended period of time under a variety of environmental loading conditions (e.g., precipitation events, seismic events, etc.). The overall objective of a communications tower is to provide the necessary elevation for antennas and associated communication components to function effectively. This objective must also be met over an extended period of time under the variety of environmental conditions expected to be encountered at the tower site over its design life.

The first step in developing a CI system is to identify the basic components of the structure. As illustrated in Figure B.1, the principal components of an earth dam might include, for example: 1) the reservoir, 2) the earth embankment, 3) the foundation system, and 4) external environmental loading factors. Together, these four relatively simple components define the much more complex structural system and the natural environment within which it has been placed.

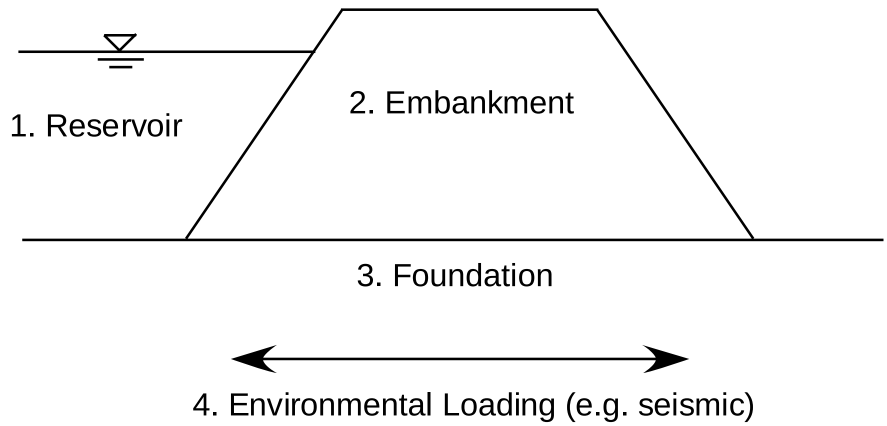


Figure B.1. Four principal components of an earth dam.

Each of these basic components of an earth dam works together in a complex, coupled fashion to satisfy the fundamental objective of retaining the reservoir. On the most basic level, the foundation supports the embankment, the embankment provides a low permeability and high strength barrier to retain the reservoir, and the height of the reservoir governs seepage through the embankment. Environmental factors (e.g., precipitation events, seismic loading) may in turn influence each of the other three basic components and thus is considered a component of the system. For example, precipitation may raise the height of the reservoir or erode the downstream slope of the embankment; an earthquake event may liquefy the foundation soils.

Assessing the overall ability of the structure to meet its design objective requires one to consider not only the physical condition of the basic structural components, but also to identify how the individual components interact through cause and effect mechanisms. In other words, if the condition of one particular component is poor, how does this affect the condition of a related component? A key consideration in developing a rational condition assessment tool, therefore, is determining how the deterioration or loss of functionality of one particular component in the system can influence the ability of the other components to fulfill their role in the larger system (i.e., a "cause" mechanism). Conversely, how does the condition or performance of any one component influence the performance of any particular component (i.e., an "effect" mechanism)? These questions may be addressed by developing what will be referred to herein as an "interaction matrix."

Step 2: Develop a component interaction matrix

Hudson (1992) proposed a generalized matrix-based approach for systematically describing complex cause and effect interactions in multi-component systems. If we consider, for example, a relatively simple system comprised of only two components, a 2 × 2 interaction matrix may be constructed to describe the cause and effect interactions between them. Figure B.2 illustrates a 2 × 2 interaction matrix for capturing the interactions between two arbitrary system components designated "A" and "B." The diagonal cells of the matrix are the principal system components. The off-diagonal cells describe the qualitative interactions between the components and are considered in a "clockwise" fashion. In other words, the cell in row 1 and column 2 (R1:C2) describes the influence of component A on component B; the cell in row 2 and column 1 (R2:C1) describes the influence of component B on component A.

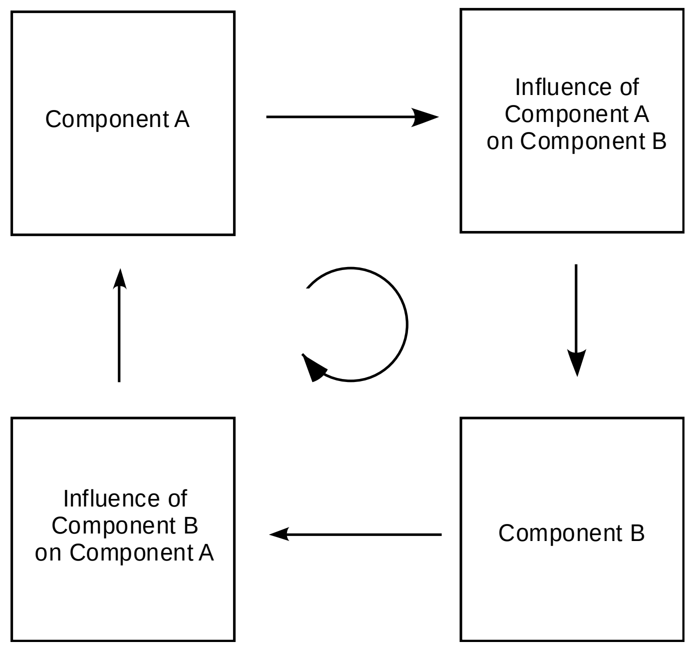


Figure B.2. 2 × 2 interaction matrix for describing cause and effect interactions between components A and B in a two-component system. The circular arrow in the center of the figure illustrates the clockwise influence convention.

Figure B.3 illustrates this concept for the earth dam system introduced earlier. If we limit the earth dam system to a two-component system consisting of an embankment and reservoir, the influence of the embankment on the reservoir is described in the upper off-diagonal cell (R1:C2). In other words, the influences the embankment has on the reservoir are to control the maximum reservoir storage capacity and volume. As shown in the lower off-diagonal cell, the influence of the reservoir on the embankment (R2:C1) is to change the effective stress in the embankment and to cause potential internal erosion (piping) via seepage processes.

Figure B.4 illustrates an example of a 4 × 4 interaction matrix for each component of a four-component earth dam system. As before, the four principal components of the overall structural system (reservoir, embankment, foundation, environmental loading) are represented in the diagonal cells of the matrix. The manner in which each of these four components interacts with the others is described by the off-diagonal cells using the same clockwise convention as in the 2 × 2 matrix. For example, the cell in row 1, column 3 (R1:C3) describes the interaction that the embankment has with the reservoir (the geometry of the embankment governs the maximum height of the reservoir). The cell R4:C1 describes the interaction that environmental loading has with the embankment (precipitation may cause surface runoff and erosion of the embankment, earthquake loading may cause liquefaction of the embankment, precipitation governs the extent of vegetative growth on the embankment). If an interaction between any two system components is considered to be insignificant, then the corresponding cell in the interaction matrix is left blank. The actual nature of the interactions between the principal components is much more complex than can be described in the cell entries, but it is an efficient mechanism for representing the entire system in an organized manner.

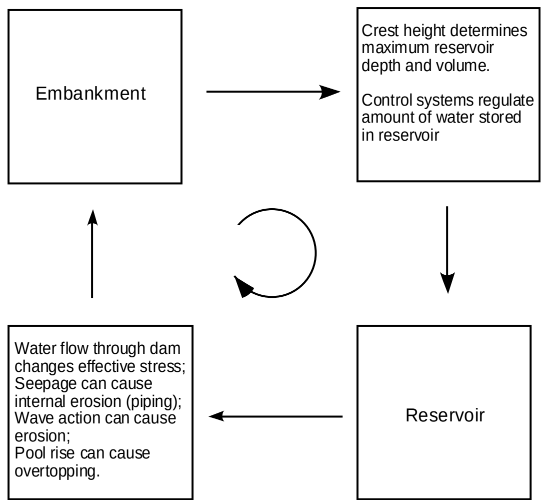


Figure B.3. 2 × 2 interaction matrix for earth dam and its reservoir (Andersen and Torrey, 1995).

COLUMN

	1	2	3	4
ROW 1	**Embankment**	consolidation due to increased load	geometry governs maximum height and volume	
2	differential settlement induces shear stress	**Foundation**	seepage could affect storage capacity	geology could affect seismic hazard
3	seepage affects state of stress and could cause piping	seepage affects state of stress and could cause piping	**Reservoir**	induced load could induce seismic events
4	surface runoff, erosion, liquefaction, vegetation	surface runoff, erosion, liquefaction, vegetation	wind waves, reservoir height, water chemistry	**Environmental Loading**

Figure B.4. 4 × 4 interaction matrix representing total embankment dam system.

The primary system components may be subdivided to more comprehensively include sub-components of the system necessary for meeting a specific condition assessment goal. For example, if the goal of condition assessment for an earth dam is specially to assess surface erosion (a critical aspect of earth dam performance), then the 4 × 4 interaction matrix might be expanded to the 9 × 9 matrix shown as Figure B.5. Here, the original “embankment” principal component has been subdivided into five sub-components that are most relevant or sensitive to surface erosion: the crest and shoulders, upstream slope, downstream slope, upstream groin area, and downstream groin area. Similarly, the “foundation” principal component has been subdivided into an abutment portion and an underlying foundation element. Although not shown here, the environmental loading principal component might also be sub-divided into particular types of environmental loading that are most likely to cause erosion (e.g., wind loading, precipitation loading). Hudson (1992) suggests that a 12 × 12 interaction matrix is about the largest size manageable.

	Column								
Row	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**
1	Crest and Shoulders	1	1	1	1			1	
2	1	Upstream Slope		1					
3	1		Dwnstrm. Slope		1				
4	1	1		Upstream Groin		1			
5	1		1		Dwnstrm. Groin	1			
6						Abutments			
7	2	1	1	2	2		Foundation		
8	2	3	1	3	1	2		Reservoir	
9	2	2	2	2	2			3	Env. loading

Figure B.5. Total system interaction matrix for assessment of earth dam surface erosion.

The diagonal cells in the first 6 rows of the matrix shown in Figure B.5 (i.e., crest and shoulders, upstream slope, downstream slope, upstream groin, downstream groin, abutments) are structural components that may be directly assessed with regard to erosion (i.e., they may be observed and rated in terms of physical condition during inspection activities). These may be referred to as “functional components.” The diagonal cells in rows 7 through 9 (foundation, reservoir, environmental loading), on the other hand, may or may not be directly observed or quantified in terms of physical condition; however, their inclusion in the interaction matrix becomes a systematic way to weight the relative importance of the various conditions of the ratable components on the overall performance of the structure. The components in these cells may be referred to as “total-system” components. A weighting procedure for systematically accounting for the physical condition of the functional components, their influences on one another, and the influence of the total-system components is described in Step 5.

The off-diagonal cells in the matrix of Figure B.5 describe how the various functional and total-system components interact in the context of a specific assessment goal. This interaction is described qualitatively as well as quantitatively. Continuing this example for assessing the condition of an earth dam with respect to surface erosion, for example, Table B.1 summarizes the relevant interactions in a qualitative sense. The relative "strength" of each of these qualitative interactions (i.e., the degree to which it may occur or is important) is quantified by assigning that cell a number ranging from zero to four, as described in Step 3.

Table B.1. Interaction descriptions for surface protective cover (erosion control) systems (Andersen and Torrey, 1995).

Row	Column	Interaction Description
1	2	Erosion at crest and shoulders can proceed downslope
1	3	Erosion at crest and shoulders can proceed downslope
1	4	Improper drainage could concentrate in groin areas
1	5	Improper drainage could concentrate in groin areas
1	8	Erosion at crest represents a loss of freeboard
2	1	Erosion gullies can progress upslope to crest
2	4	Erosion can cross over into groin areas
3	1	Erosion gullies can progress upslope to crest
3	5	Erosion can cross over into groin areas
4	1	Erosion gullies can progress upslope to crest
4	2	Erosion gully can lead to slope instability
4	6	Erosion gully can lead to slope instability
5	1	Erosion gullies can progress upslope to crest
5	3	Erosion gully can lead to slope instability
5	6	Erosion gully can lead to slope instability
6	4	Improper drainage could concentrate in groin areas
6	5	Improper drainage could concentrate in groin areas
7	1	Differential settlement could lead to surface cracking and subsequent erosion
7	2	Differential settlement could lead to surface cracking and subsequent erosion
7	3	Differential settlement could lead to surface cracking and subsequent erosion
7	4	Differential settlement could lead to surface cracking and subsequent erosion
7	5	Differential settlement could lead to surface cracking and subsequent erosion
8	1	A high pool and large waves could lead to overtopping
8	2	Large waves could displace riprap and cause erosion
8	3	A high pool could raise piezometric water level in embankment and result in erosive seepage
8	4	Large waves could displace riprap and cause erosion
8	5	A high pool could raise piezometric water level in embankment and result in erosive seepage
8	6	Storm waves can cause erosion
9	1	Rain and wind cause erosion. Drought can result in desiccation cracking, which can lead to erosion
9	2	Freezing and thawing can degrade riprap and increase erosion potential
9	3	Rain can cause surface runoff erosion. Drought can result in desiccation cracking which can lead to erosion
9	4	Rain can cause surface runoff erosion.
9	5	Rain can cause surface runoff erosion.
9	8	Heavy rains may raise the pool. High winds may generate large waves

Step 3: Code the interaction matrix for interaction strength

The numerical values in the off-diagonal cells of the interaction matrix designate the relative strength of the interactions between the functional and total system components. The interactions follow the clockwise interaction convention introduced previously. For example, the cell (R8:C2) in Figure B.5 describes the influence of the reservoir on the upstream slope of the embankment. This interaction was identified qualitatively in Table B.1 as: "large waves [from the reservoir] could displace riprap [on the upstream slope] and cause erosion [to the upstream slope]." The strength of this particular interaction is assigned a numerical value on a scale from zero to four, in this case three. As summarized on Table B.2, this corresponds to a "strong" interaction.

As noted by Andersen and Torrey (1995), interaction strength values are ideally assigned by analytical expressions or numerical algorithms and their strengths directly compared upon some uniform basis. This would give the most detailed representation of the actual nature of the interactions, but is impractical in most cases. Moreover it is not possible to accurately quantify many of the interactions without fundamental research. Interaction strengths are therefore more commonly assigned using the engineering judgment of experts familiar with the overall behavior of the system under consideration. Hudson (1992) refers to this as "expert semi-quantitative" (ESQ) coding.

Table B.2. Interaction strength levels for expert semi-quantitative (ESQ) coding.

Qualitative Description of Interaction Strength	Numerical Value
No significant interaction	0
Weak interaction	1
Medium interaction	2
Strong interaction	3
Critical interaction	4

Once the individual cells of the interaction matrix are coded for interaction strength, the interaction matrix can be interpreted in terms of cause and effect. The goal of this interpretation is to quantify the relative dominance of any one particular functional or total-system component. The physical condition of the dominant functional components may then be heavily weighted toward assessing the overall physical condition of the multi-component structure.

For any particular component in the interaction matrix (i.e., for any diagonal cell), all of the off-diagonal cells contained in its row describe how that component influences the rest of the system (cause). Similarly, all of the off-diagonal dells contained in its column describe how the other components in the system influence it (effect). Dominant components are those that have the greatest influence on the rest of the system. Subordinate components are those that are most influenced by the rest of the system. Because the off-diagonal cells in the matrix have each been assigned a numerical value to reflect the strength of that particular interaction, the relative dominance or subordinance of any particular component can be quantified. In other words, the sum of the numbers in the row, or "cause score," of a dominant system component is a relatively large number. The sum of the numbers in the column, or "effect score," of a subordinate system component is a relatively large number. These scores are used subsequently to develop weighting factors for the physical condition of each component in the system (Step 5).

Step 4: Define ranges between ideal and failed conditions for each component

To develop an overall condition rating for a structure, the individual functional components of the system must be quantitatively rated in a rational, repeatable, and relatively universal manner. To reduce subjectivity often associated with quantitative rating of functional system components, focus can be placed on considering deviations from ideal and failed conditions. The condition of any particular component may then be assigned a value from 0 to 100 to reflect deviation from the ideal condition. Ideal and failed conditions for each component, however, must first be defined. Table B.3, for example, shows Andersen and Torrey's (1995) definitions for ideal and failed conditions for the functional components of an earth dam with respect to surface erosion. Table B.4 delineates a corresponding indexing scale for quantifying the physical condition of the functional component under consideration.

Table B.3. Definitions of ideal and failed conditions for functional components of earth dam in the context of erosion.

System Subunit	Ideal condition
Crest and Shoulders	100% coverage of surface with roadway material or apron material
Upstream Slope	100% coverage with vegetation or riprap
Downstream Slope	100% coverage with vegetation or riprap
Upstream Groin	100% coverage with riprap or original material
Downstream Groin	100% coverage with riprap or original material

Table B.4. Indexing scale for quantifying condition of system components.

Condition Index	Condition Description
85 – 100	Excellent: No noticeable deviation from ideal condition
70 – 84	Very Good: Only slight deviations from the ideal condition are evident
55 – 69	Good: Some deviation from the ideal condition evident but function is not significantly affected
40 – 54	Fair: Moderate deviation from the ideal condition evident but function is adequate
25 – 39	Poor: Serious deviation from ideal condition in at least some portion of the component; function is inadequate
10 – 24	Very Poor: Extensive deviation from ideal condition: Component is barely functional
0 – 9	Failed: All or a potion of component is missing or has failed

Step 5: Develop weighting factors and formulate condition index scalar

The final step in producing an overall condition index for a complex multi-component system is to systematically "weight" the component physical conditions assigned in Step 4 to the overall condition of the structure. This may be done in a simple linear fashion by considering the "cause" and "effect" scores of the individual components identified in the coded interaction matrix (Step 3). Specifically, we can define a "total" score for a particular component as the sum of its cause and effect score. A weighting factor for the numerical condition of any one component may then be defined as the ratio of that component's "total" score to system's total score. The weighted conditions for each component may then be summed to generate an overall condition index for the structure.

These concepts are demonstrated on Table B.5 for the earth dam interaction matrix shown previously as Figure B.5. Note that of the six functional components, the crest and shoulders are the most interactive (highest weighting factor, 0.24) and the abutments are the least interactive (lowest weighting factor, 0.09). Thus, the physical condition of the crest and shoulders has a proportionately greater contribution to the overall condition of the earth dam system, which may be computed as follows:

$$CI_{ed} = CI_{cs}(0.24) + CI_{us}(0.16) + CI_{ds}(0.13) + CI_{ug}(0.21) + CI_{dg}(0.17) + CI_{a}(0.09) \tag{2.1}$$

where CI_{ed} is the overall condition index of the earth dam, CI_{cs} is the component condition index of the crest and shoulders, CI_{us} is the component condition index of the upstream slope, CI_{ds} is the component condition index of the downstream slope, CI_{ug} is the component condition index of the upstream groin, CI_{dg} is the component condition index of the downstream groin, and CI_{a} is the component condition index of the abutments. Note that the sum of the weighting factors is one.

Finally, as indicated on Table B.6, the overall condition index for the structure ($0 < CI < 100$), may be correlated to a qualitative description which may in turn be used as a rational basis for recommended action and a corresponding basis for the managing agency to allocate funds for repair, evaluation, maintenance, and rehabilitation (REMR) activities.

Table B.5. Development of weighting factors from earth dam interaction matrix.

Subunit	Cause Score	Effect Score	Total Score	Weight	Weight Factor
Crest and Shoulders	5	10	15	15/63	0.24
Upstream Slope	2	8	10	10/63	0.16
Downstream Slope	2	6	8	8/63	0.13
Upstream Groins	3	10	13	13/63	0.21
Downstream Groins	3	8	11	11/63	0.17
Abutments	2	4	6	6/63	0.09
Total	17	46	63	---	---

Table B.6. USACE condition indexing scale for earth dams.

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	

Appendix C. Simulation of Ground Motions Compatible with Response Spectra

(Summarized by Jianhua Li, University of Missouri – Columbia)

Temporal and spatial variability are two important aspects of earthquake loads. For structures with relatively small footprints, such as conventional buildings or communications towers, the spatial variability has little influence on their seismic responses and ground motions considering only temporal variability are generally adopted.

Temporal ground motions for the dynamic structural analyses presented in this report were developed from frequency response spectra generated using the simplified UBC procedure. Procedures used to simulate ground motions from the frequency response spectra are summarized in the following.

Basic model

Strong ground motion generally shows three stages: increasing rapidly from weak to strong; maintaining its average strength; and gradually decreasing. Accordingly, the basic model of ground motion can be expressed by

$$z(t) = f(t)x(t) \tag{C1}$$

where $f(t)$ is the strength envelope function. In general, it has the following form (See Figure C1):

$$f(t) = \begin{cases} (t/t_1)^2, & t \le t_1 \\ 1, & t_1 < t \le t_2 \\ e^{-c(t-t_2)}, & t > t_2 \end{cases} \tag{C2}$$

in which t_1, t_2, c are the envelope parameters. t_1, t_2 indicate the starting and finishing time of stationary stage, c controls the decline rate of the attenuation stage. Their values can be obtained by statistical analysis of recorded ground motions or by random vibration theory (Jennings & Housner, 1968; Huo & Hu, 1991; Ou & Niu, 1991).

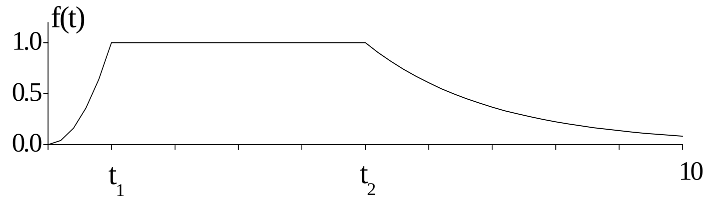


Fig. C1 Strength envelope function

The trigonometric series is the most popular way to express $x(t)$, namely

$$x(t) = \sum_{k=0}^{N-1} c_k \cdot \cos(\omega_k t + \varphi_k) \tag{C3}$$

where N is the total number of the frequency components; c_k and ω_k denote the amplitude and frequency corresponding to the kth frequency component; φ_k is the phase angle, taken as uniform but randomly distributed in the range $(0, 2\pi)$ (Ohsaki, 1979). Shinozuka & Deodatis (1991) pointed out that $x(t)$ is an asymptotically Gaussian process as $N \to \infty$. From Eq. 3, the mean squares of $x(t)$ can be given by

$$E\left[x^2(t)\right] = \sum_{k=0}^{N-1} c_k^2 E\left[\cos^2(\omega_k t + \varphi_n)\right] = \sum_{k=0}^{N-1} \frac{1}{2} c_k^2 \tag{C4}$$

Assuming $S(\omega)$ the power spectral density of $x(t)$, then the mean squares also equals

$$E\left[x^2(t)\right] = 2\int_0^{+\infty} S(\omega) d\omega = 2\sum_{k=0}^{N-1} S(\omega_k)\Delta\omega \tag{C5}$$

From Eq. 4 and Eq. 5, we can find that c_k can be expressed in terms of $S(\omega)$,

$$c_k = \sqrt{4S(\omega_k)\Delta\omega} \tag{C6}$$

During the simulation process, ω_k, $\Delta\omega$ are taken as

$$\omega_k = 2\pi k/T_d\ ,\ \Delta\omega = 2\pi/T_d \tag{C7}$$

in which T_d is the duration of ground motion.

The cost of digitally generating ground motions shown in Eq. 3 can be drastically reduced by using the Fast Fourier Transform (FFT) technique (e.g. Brigham, 1988). In order to take advantage of the FFT technique, Eq. 3 is rewritten in the following form:

$$x(t) = \mathrm{Re}\left\{\sum_{k=0}^{M-1} B_k \exp(\mathrm{i}2\pi kp/M)\right\} \quad p = 0,1,\cdots,M-1 \tag{C8}$$

where $M = 2^\eta$ (η is a positive integer) and $M \geq 2N$;

$$B_k = c_k \exp(\mathrm{i}\varphi_k) \tag{C9}$$

and

$$B_0 = 0 \tag{C10}$$

$$B_k = 0, \quad \text{for } N \le k \le M-1 \tag{C11}$$

Transformation between power spectral density and mean response spectrum

From Eq. 4, it may be seen that the power spectral density of ground motion is required to compute c_k. In order to synthesize the ground motion compatible with the given response spectrum, it is required to find the power spectral density (PSD) consistent with a given response spectrum using the transformation relation between them. As shown in Figure B2, a single-degree-of-freedom oscillator with the natural frequency ω_n and the damping ratio ζ, is subjected to a random process $z(t)$.

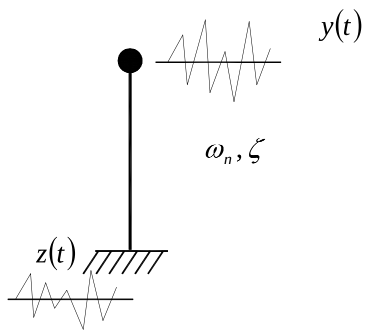


Fig.C2 Single-degree-of-freedom oscillator

If we use $y(t)$ to denote its response and y_m denote the peak absolute values of $y(t)$, then based on random vibration theory, y_m has the cumulative distribution as below

$$F_y(y_m) = \exp\left[-\nu T_d \exp\left(-\frac{y_m^2}{2\sigma_y^2}\right)\right] \tag{C12}$$

where T_d is the duration; ν is the mean zero-crossing rate given by $\nu = \omega_n / \pi$; and σ_y is the root-mean-square of the response $y(t)$. From Eq. (12), we can get the mean of y_m as below (Davenport, 1964)

$$\mu_{y_m} = \left(\sqrt{2\ln \nu T_d} + \frac{0.5772}{\sqrt{2\ln \nu T_d}}\right)\sigma_y \tag{C13}$$

If we use $S(\omega)$ to denote the PSD of ground motion, $S_y(\omega)$ to denote the PSD of $y(t)$, then

$$S_y(\omega) = \left|H(\mathrm{i}\omega)\right|^2 S(\omega) \tag{C14}$$

in which $H(\mathrm{i}\omega)=1/\left(\omega_n^2-\omega^2+2\mathrm{i}\zeta\omega_n\omega\right)$ is the frequency response function. According to the definition of σ_y, then

$$\sigma_y^2=\int_{-\infty}^{+\infty}S_y(\omega)d\omega=\int_{-\infty}^{+\infty}\left|H(\mathrm{i}\omega)\right|^2S(\omega)d\omega \tag{C15}$$

If $z(t)$ are the wide-band inputs, Eq.15 can be reduced to

$$\sigma_y^2=\frac{\pi\omega_nS(\omega_n)}{2\zeta} \tag{C16}$$

A response spectrum gives the maximum response to certain ground motion for single DOF oscillators with different natural periods and damping ratios. Design response spectra specified in design codes are generally mean response spectra given by averaging the response spectra corresponding to a large amount of recorded ground motions. In this sense, the mean response spectrum $S_a(\omega,\xi)$ is the mean of the peak absolute response, e.g. μ_{y_m} in Eq.13. Then from Eq. 13 and Eq. 16, we can get the transformation relation between $S(\omega)$and $S_a(\omega,\xi)$, namely

$$S(\omega)=\frac{2\zeta}{\pi\omega}S_a^2(\omega,\xi)\Bigg/\left(\sqrt{2\ln\nu T_d}+\frac{0.5772}{\sqrt{2\ln\nu T_d}}\right)^2 \tag{C17}$$

or from Kaul (1978):

$$S(\omega,\zeta)=\sqrt{-2\ln\left(-\frac{\pi\ln\rho}{\omega T_d}\right)}\left(\frac{\pi\omega S(\omega)}{\zeta}\right)^{1/2} \tag{C18}$$

where ρ is the probability of exceedance. Similarly, Jiang & Hong (1984) suggested

$$S(\omega)=\frac{2\xi}{\pi\omega}S_a^2(\omega,\xi)\Bigg/M\left(\sqrt{2\ln\nu T_d}+\frac{0.5772}{\sqrt{2\ln\nu T_d}}\right)^2 \tag{C19}$$

where

$$M=t_2-0.835t_1+\frac{3}{8c}+\frac{1}{2\xi\omega}\left(e^{-2\xi\omega t_2}-e^{-2\xi\omega t_1}\right)+\frac{e^{-2\xi\omega t_2}}{2(c+\xi\omega)}\left[e^{-2(c+\xi\omega)\ln 2/c}-1\right]$$

and Kiureghian & Neuenhofer (1991) suggested:

$$S(\omega)=\frac{1}{\omega^2}\left(\frac{2\zeta\omega}{\pi}+\frac{4}{\pi T_d}\right)S_a^2(\omega,\xi)\Bigg/\left(\sqrt{2\ln \nu T_d}+\frac{0.5772}{\sqrt{2\ln \nu T_d}}\right)^2 \qquad \text{(C20)}$$

Simulation procedure

The basic procedure for simulating ground motion can be summarized as follows:

Determine the target response spectrum $S_a(\omega,\zeta)$ (usually, the design response spectrum), and the strength envelope function $f(t)$. Select the number (K) of ordinates of $S_a(\omega,\xi)$ to be controlled within given allowable error ε. Generally, K is around 40-100 with the corresponding periods approximately located with constant frequency intervals, and ε =5-10%.

Generate an initial time history $z_0(t)$ by using Eq. 6 - Eq. 11, where $S(\omega_k)$ should be transformed from $S_a(\omega_k,\zeta)$ by using Eq. 18, Eq. 19 or Eq. 20; φ_k is taken a value in the range $(0,2\pi)$ by the random sampling.

Calculate the response spectrum $S_a^c(\omega_k,\zeta)$ of $z_0(t)$, the calculated spectrum. Since the above transformation relation is approximate, the relative errors between the values of the calculated spectrum and of the target spectrum at some controlling ordinates are greater than the allowable error ε. Figure C3 shows an example to indicate this point.

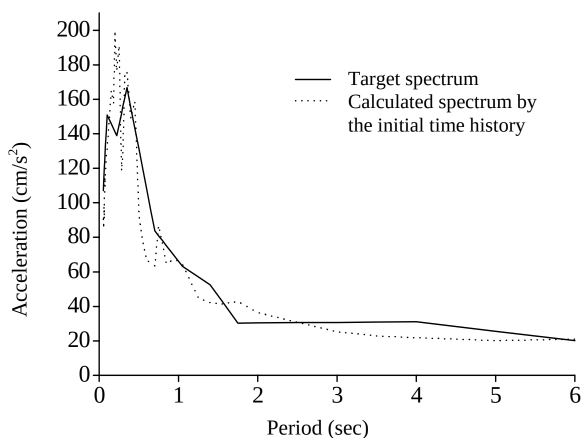


Fig.C3 Comparison of target spectrum and calculated spectrum by the initial time history

A fitting technique should be used to modify the initial time history to iteratively obtain a new time history until all controlling ordinates of the calculated response spectrum close to the target within the error allowed. The ordinary method is to make a modification of Fourier amplitudes c_k by

$$c_k^{m+1} = \frac{S_a(\omega_k, \zeta)}{S_a^c(\omega_k, \zeta)} c_k^m \tag{C21}$$

where c_k^m are the results at mth iteration, c_k^{m+1} are the results at $(m+1)$th iteration. Sometimes the ordinary method isn't very efficient, especially for the ordinates with long periods. Improved fitting techniques (e.g. Unruh & Kana, 1981; Hu & He, 1986) could be adopted at this time. Figure C4 gives an example to indicate how well the calculated spectrum corresponding to the final time history matches the target spectrum by using improved fitting technique.

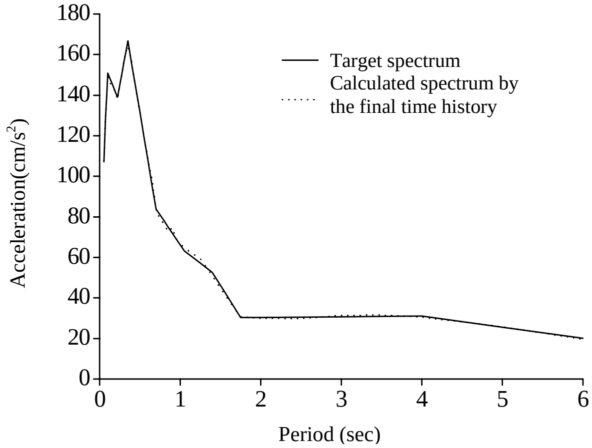


Fig.C4 Comparison of target spectrum and calculated spectrum by the initial time history

Numerical example

Consider a target response spectrum developed using procedures from the *Uniform Building Code* (1997) for a site with soil profile type of rock (S_A) in seismic zone 2A. The UBC spectrum is shown in Figure C5.

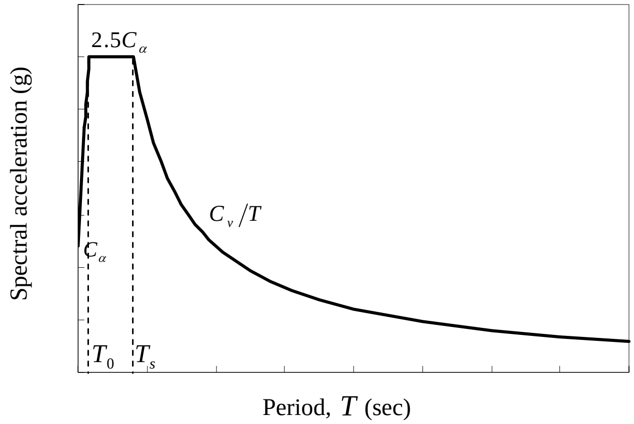


Fig.C5 Example UBC response spectrum

And the associated parameters are:

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

The parameters of simulated ground motion are:

$t_1 = 1\text{s}$, $t_2 = 9\text{s}$, $c = 0.2$
$T_d = 15\text{s}$, $a_{\max} = C_a = 0.12\text{g} = 117.6\text{cm/s}^2$ (Peak ground acceleration)

The number of controlling periods K equals 41, the allowable error ε =5%.

Figure C6 compares the target UBC spectrum and the spectrum calculated from the simulated ground motion. It is clear that the calculated spectrum agrees with the target one very well. Figure C7 shows the simulated acceleration, velocity and displacement time history, respectively.

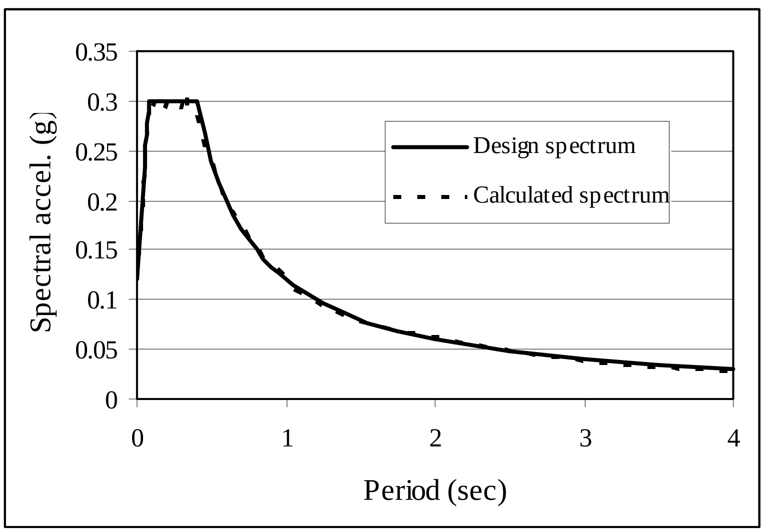
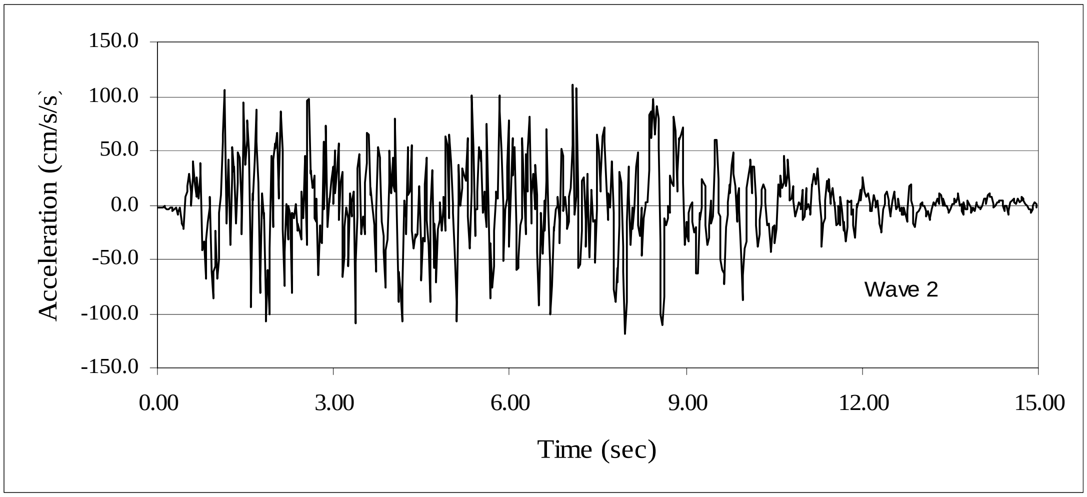
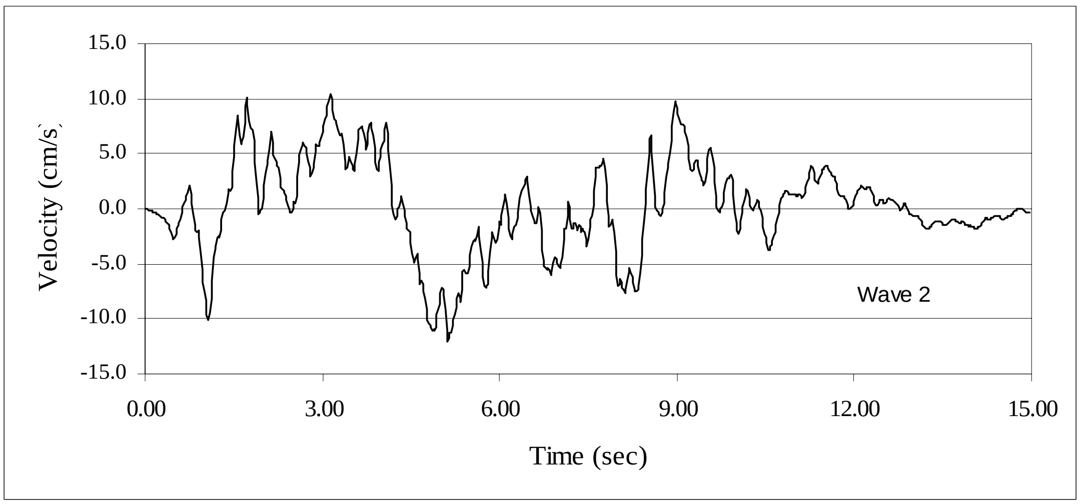


Fig.C6 Comparison of design and calculated spectrum





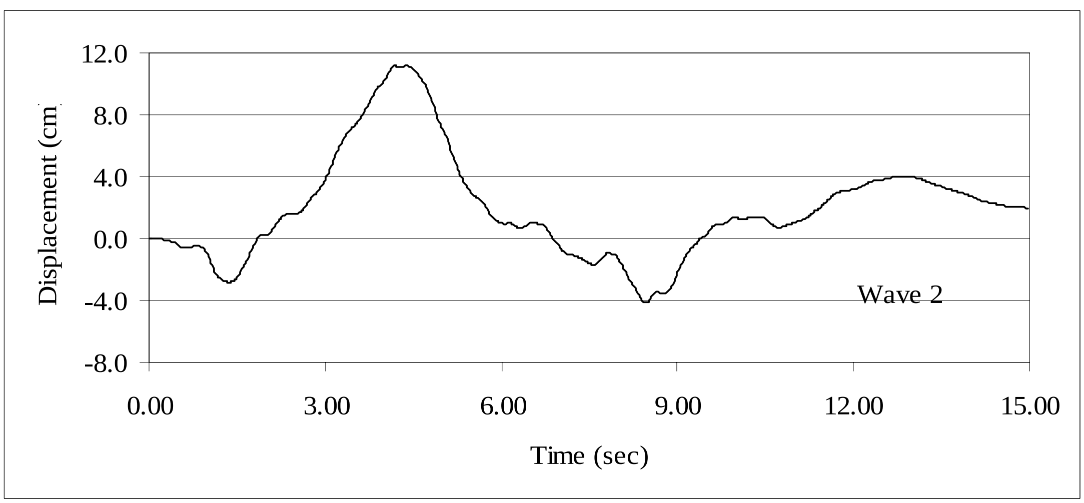
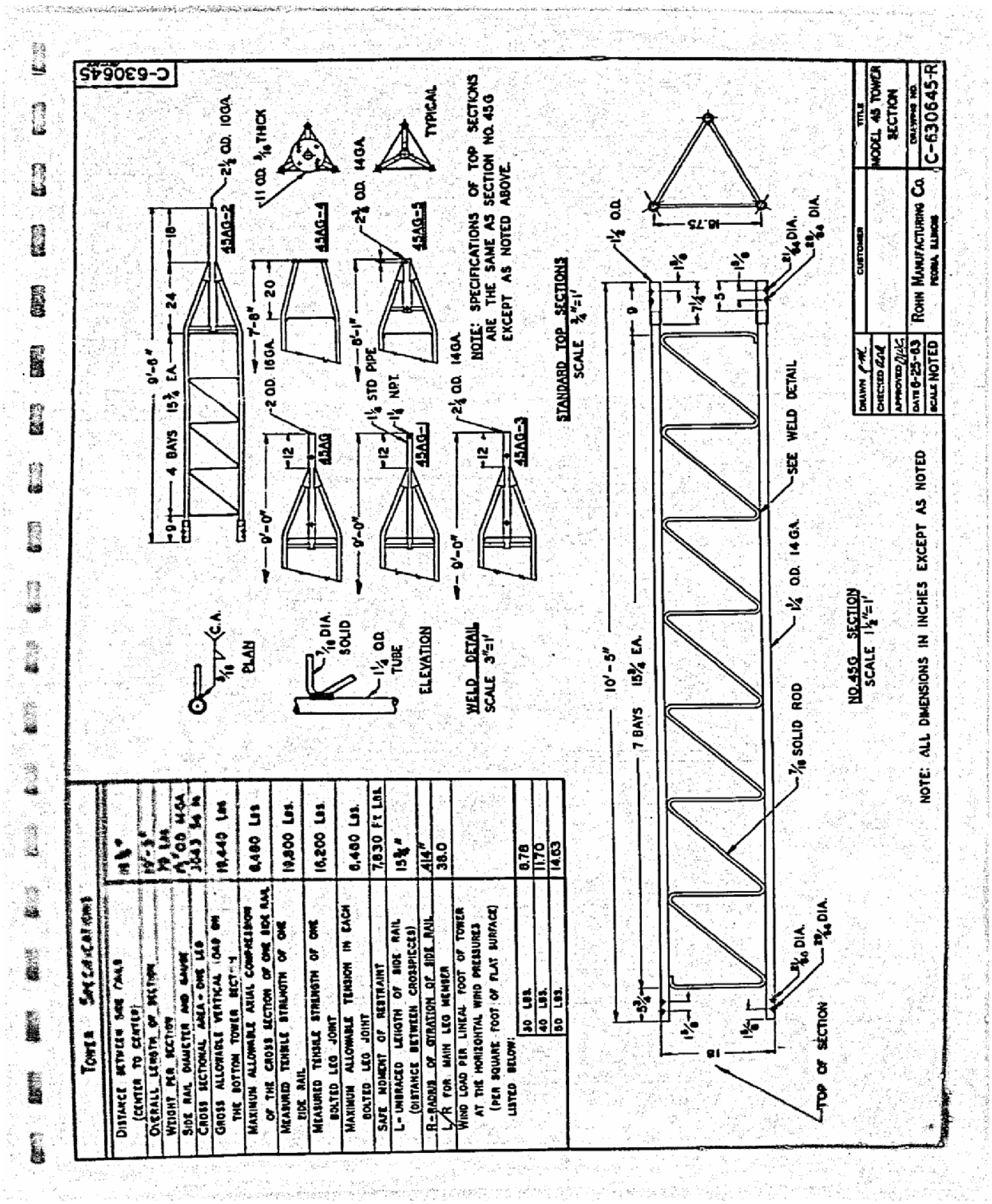
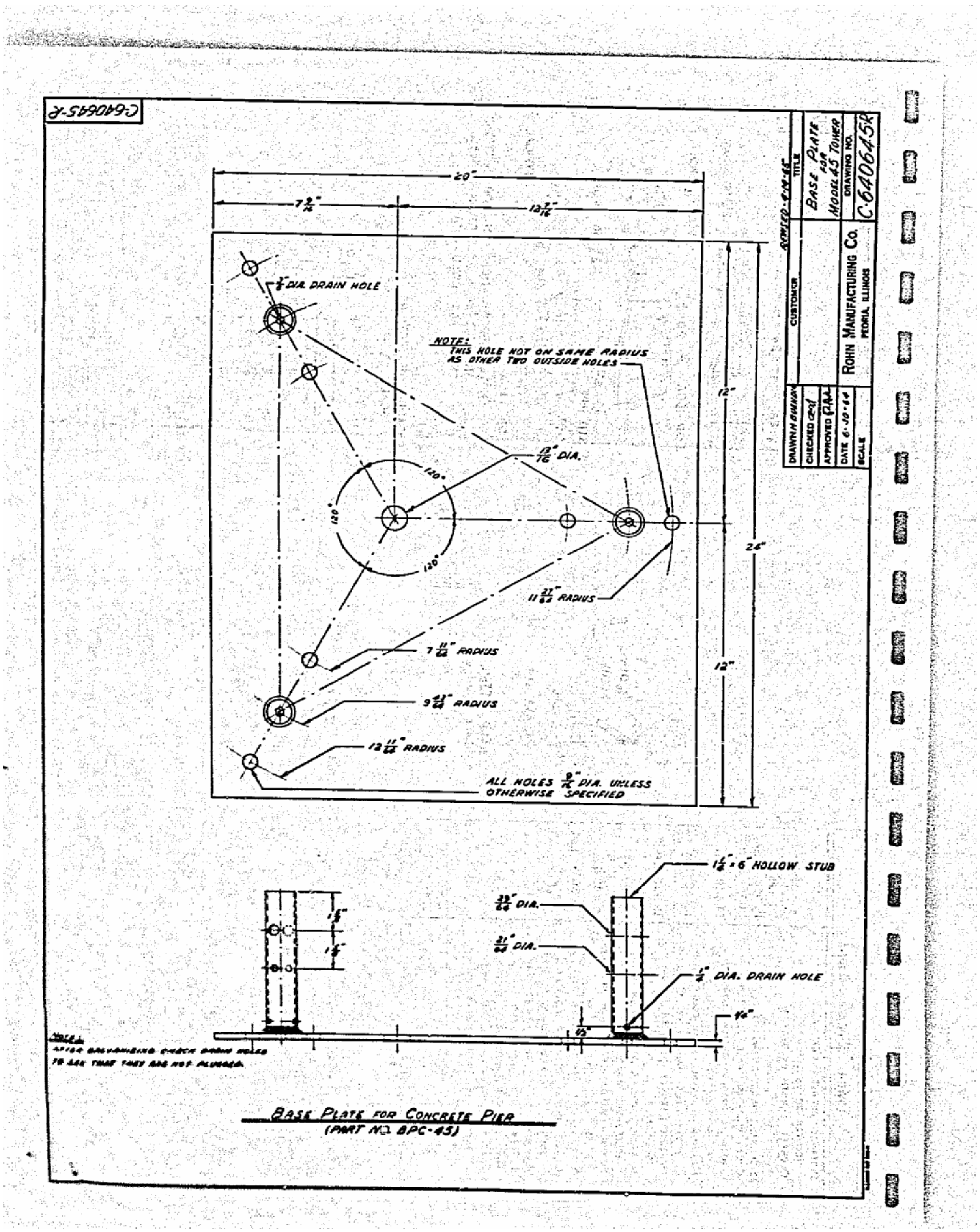
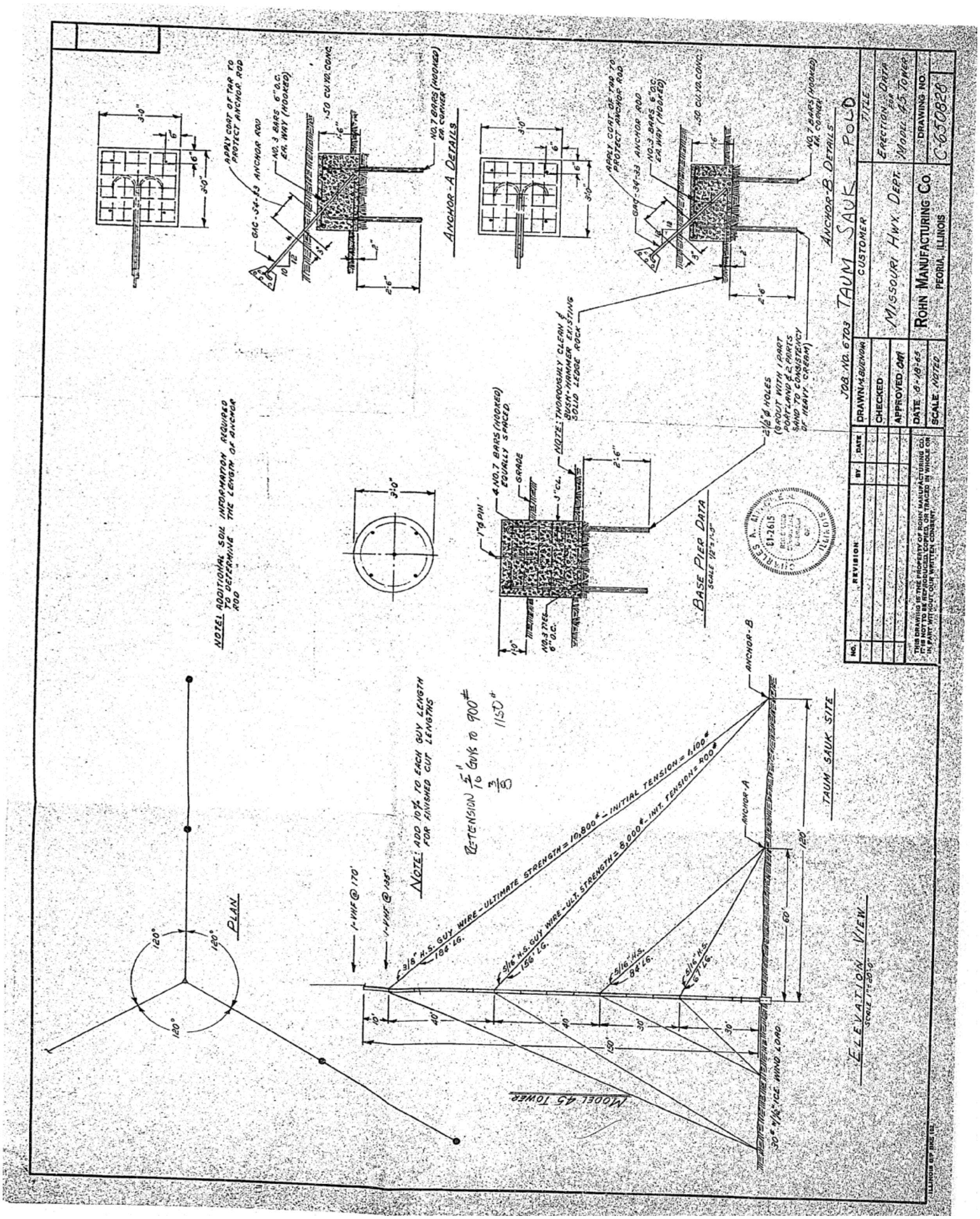


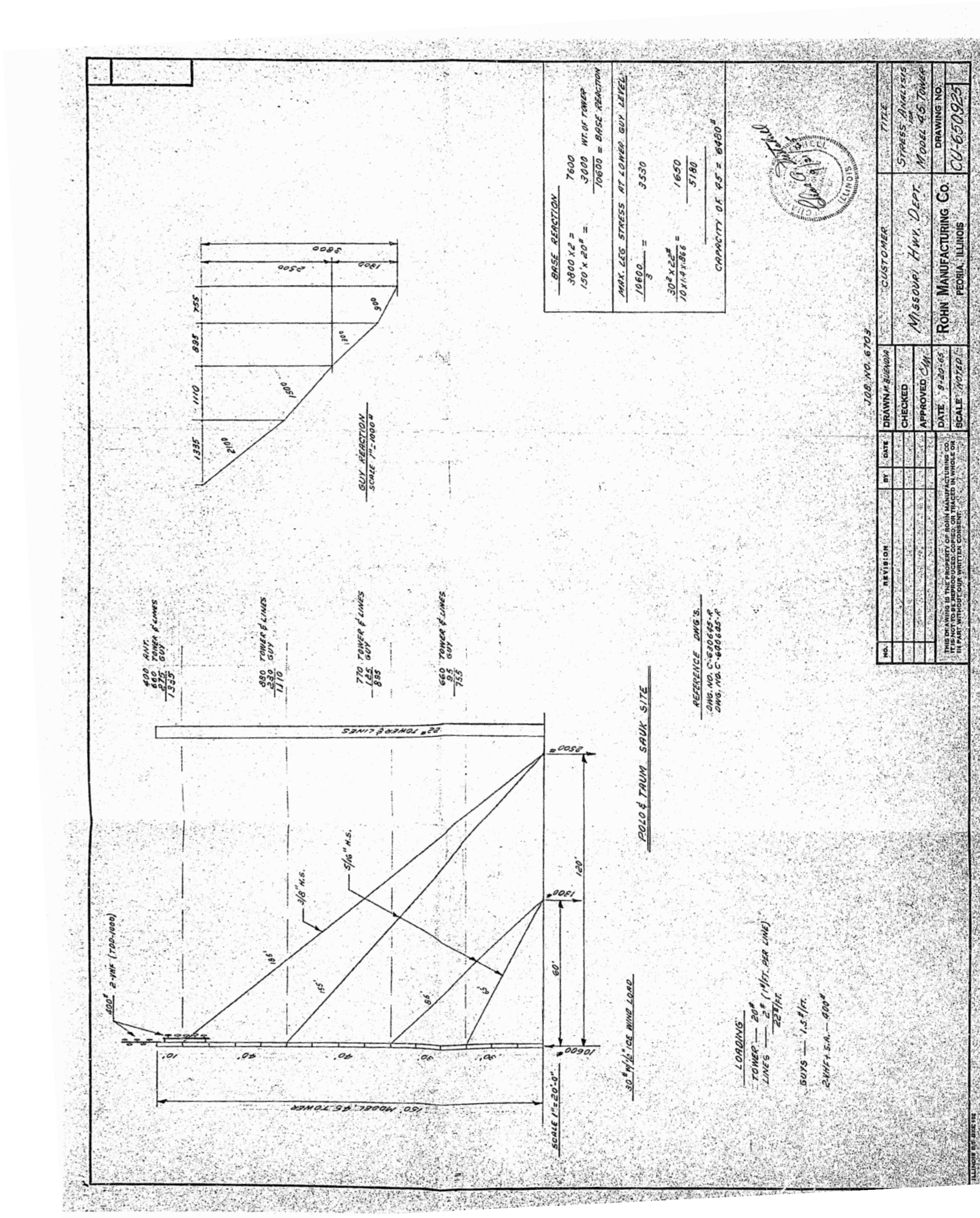
Fig.C7 Simulated acceleration, velocity and displacement time history

Appendix D. Structural Drawings for Taum Sauk Tower



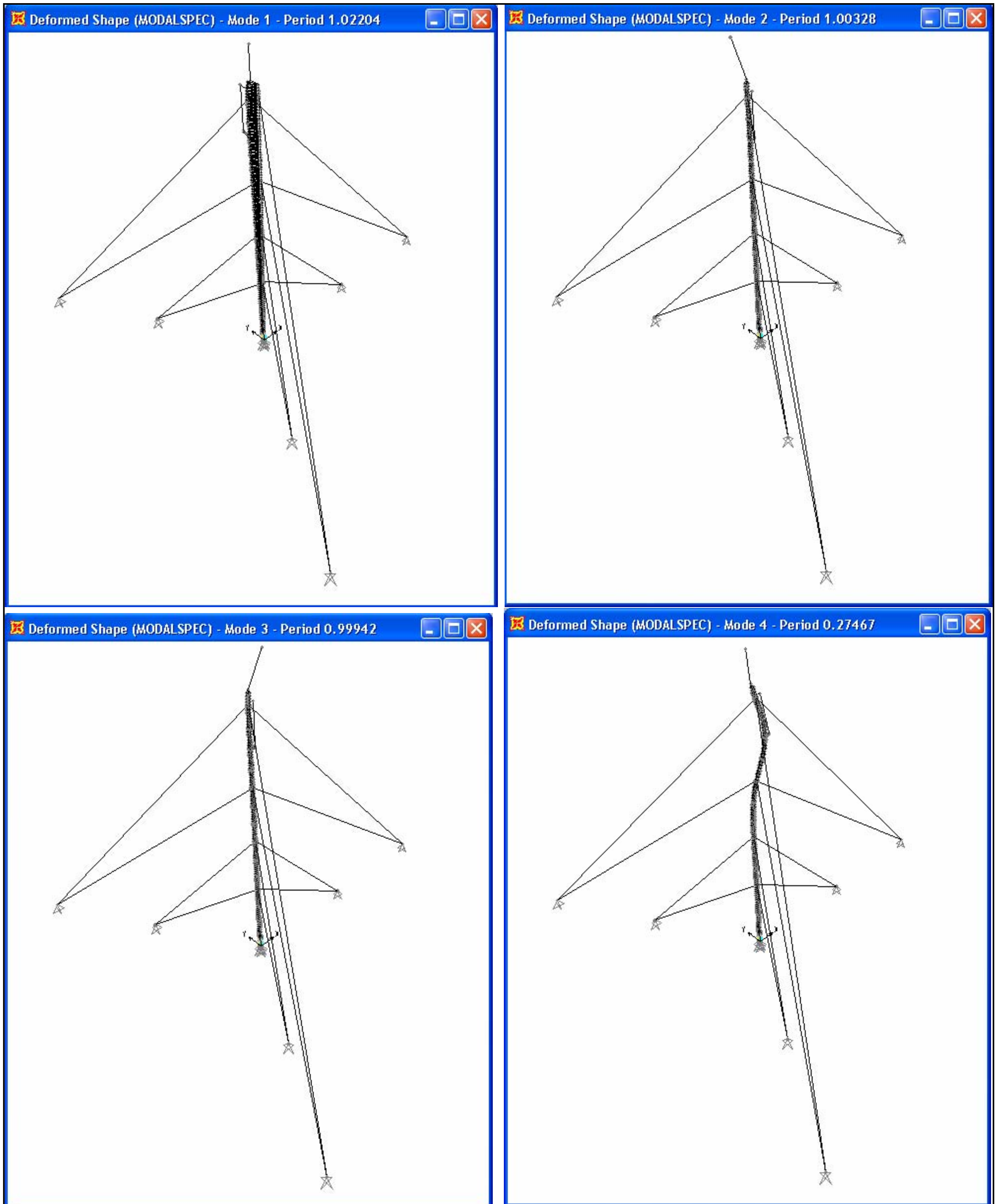

TITLE
BASE PLATE FOR MODEL 45 TOWER
DRAWING NO.
C-640645R
CUSTOMER
ROHN MANUFACTURING CO.
PEORIA, ILLINOIS
DRAWN
CHECKED
APPROVED
DATE
SCALE
20"
NOTE:
THIS HOLE NOT ON SAME RADIUS AS OTHER TWO OUTSIDE HOLES
DIA. DRAIN HOLE
DIA.
RADIUS
120°
12"
24"
ALL HOLES DIA. UNLESS OTHERWISE SPECIFIED
HOLLOW STUB
DIA. DRAIN HOLE
BASE PLATE FOR CONCRETE PIER
(PART NO. BPC-45)


PLAN
ELEVATION VIEW
MODEL 45 TOWER
TAUM SAUK SITE
ANCHOR-A
ANCHOR-B
BASE PIER DATA
ANCHOR-A DETAILS
ANCHOR-B DETAILS
JOB NO. 6703
MISSOURI HWY. DEPT.
ROHN MANUFACTURING CO.
PEORIA, ILLINOIS
C650828

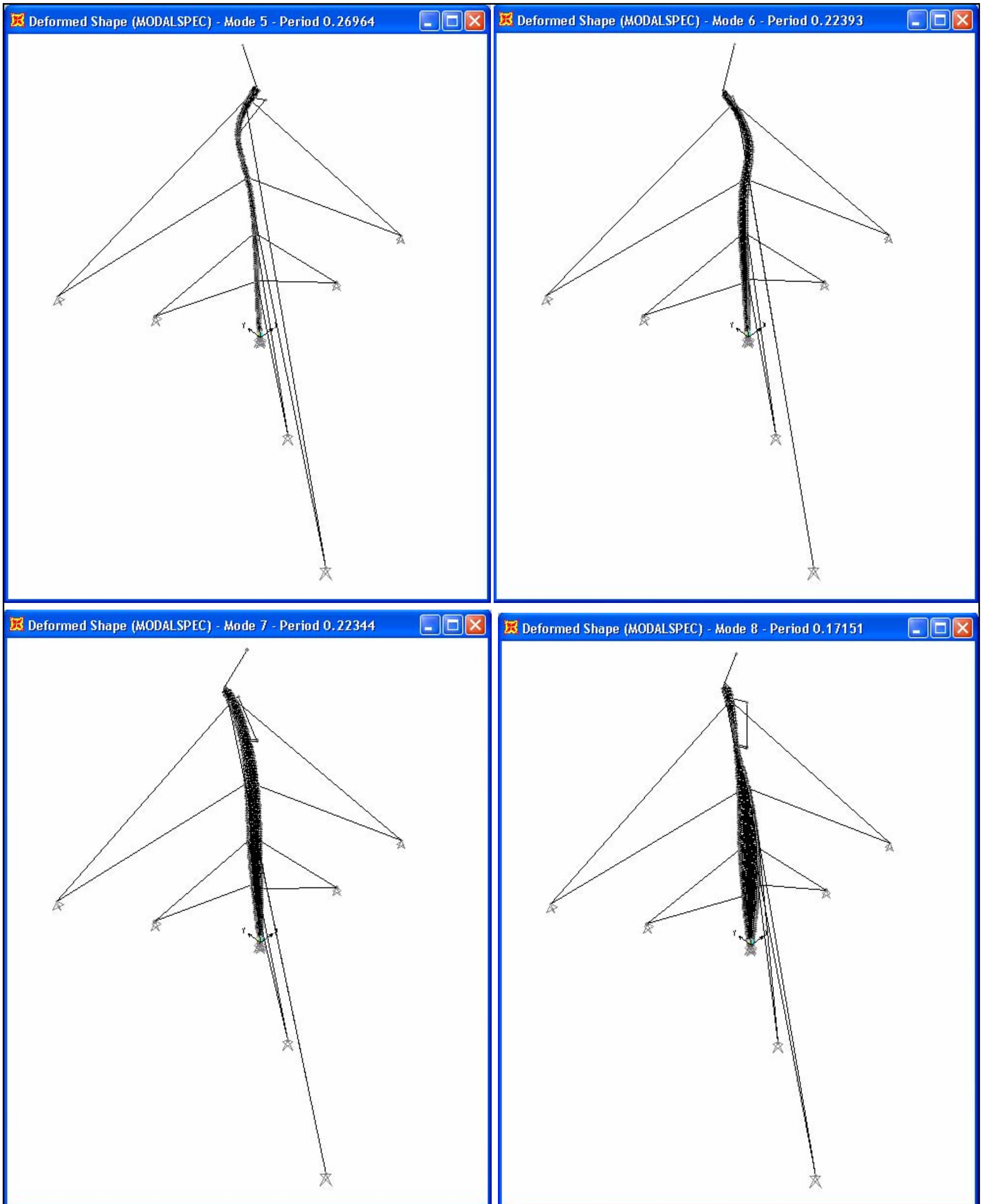

POLO & THUM SAUK SITE
GUY REACTION
SCALE 1" = 1000#
BASE REACTION
3800 X 2 = 7600
150' x 20# = 3000 WT. OF TOWER
10600 = BASE REACTION
MAX. LEG STRESS AT LOWER GUY LEVEL
CAPACITY OF 45 = 6480#
150' MODEL 45 TOWER
SCALE 1" = 20'-0"
LOADING
JOB NO. 6703
DRAWN M. BUENDIA
CHECKED
APPROVED
DATE 9-20-65
SCALE NOTED
CUSTOMER
MISSOURI HWY. DEPT.
TITLE
STRESS ANALYSIS FOR MODEL 45 TOWER
ROHN MANUFACTURING CO.
PEORIA, ILLINOIS
DRAWING NO.
CU-650925
THIS DRAWING IS THE PROPERTY OF ROHN MANUFACTURING CO. IT IS NOT TO BE REPRODUCED, COPIED, OR TRACED IN WHOLE OR IN PART WITHOUT OUR WRITTEN CONSENT.

Appendix E. Modes of Vibration for Taum Sauk Tower

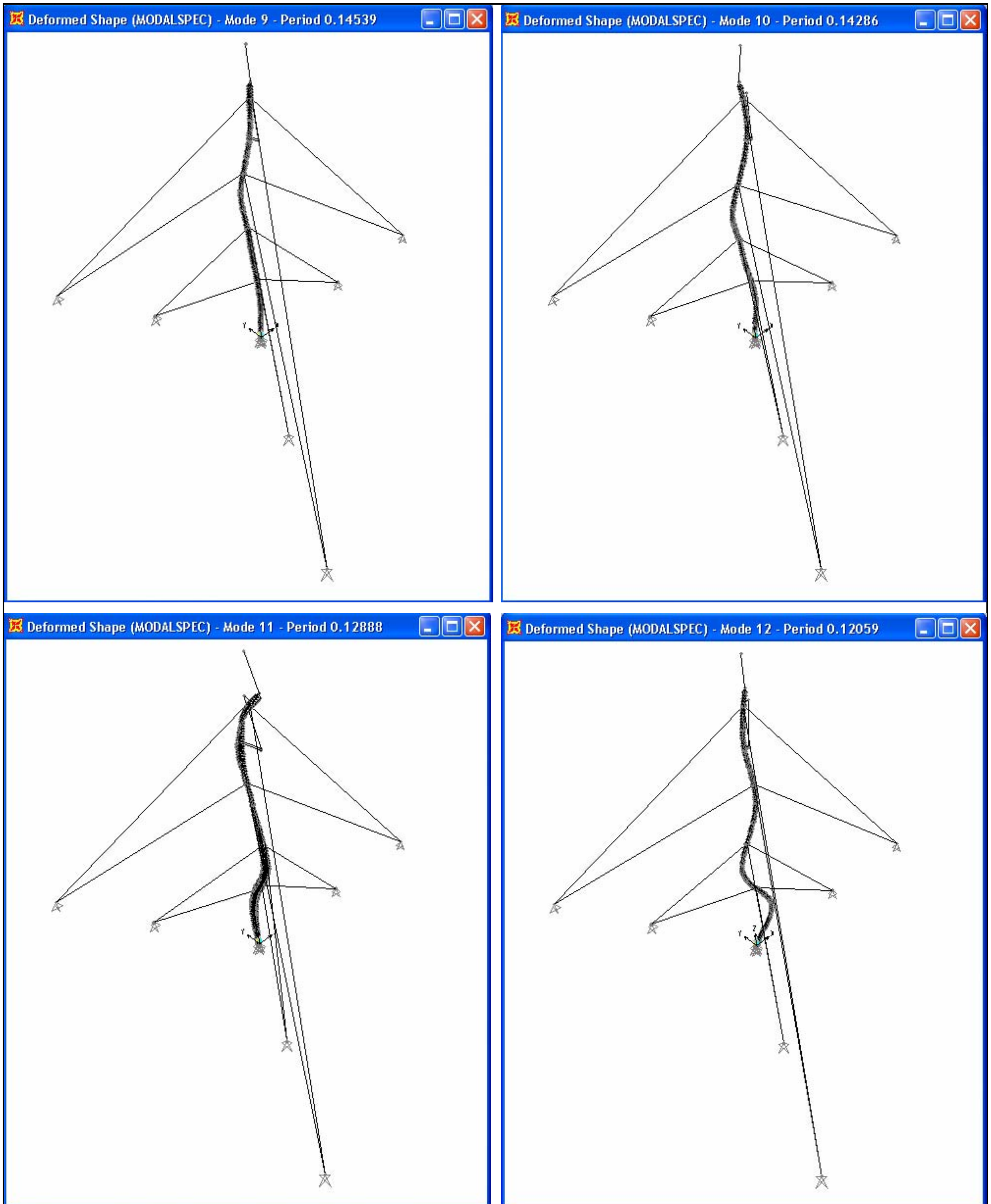
1. Mode Shapes



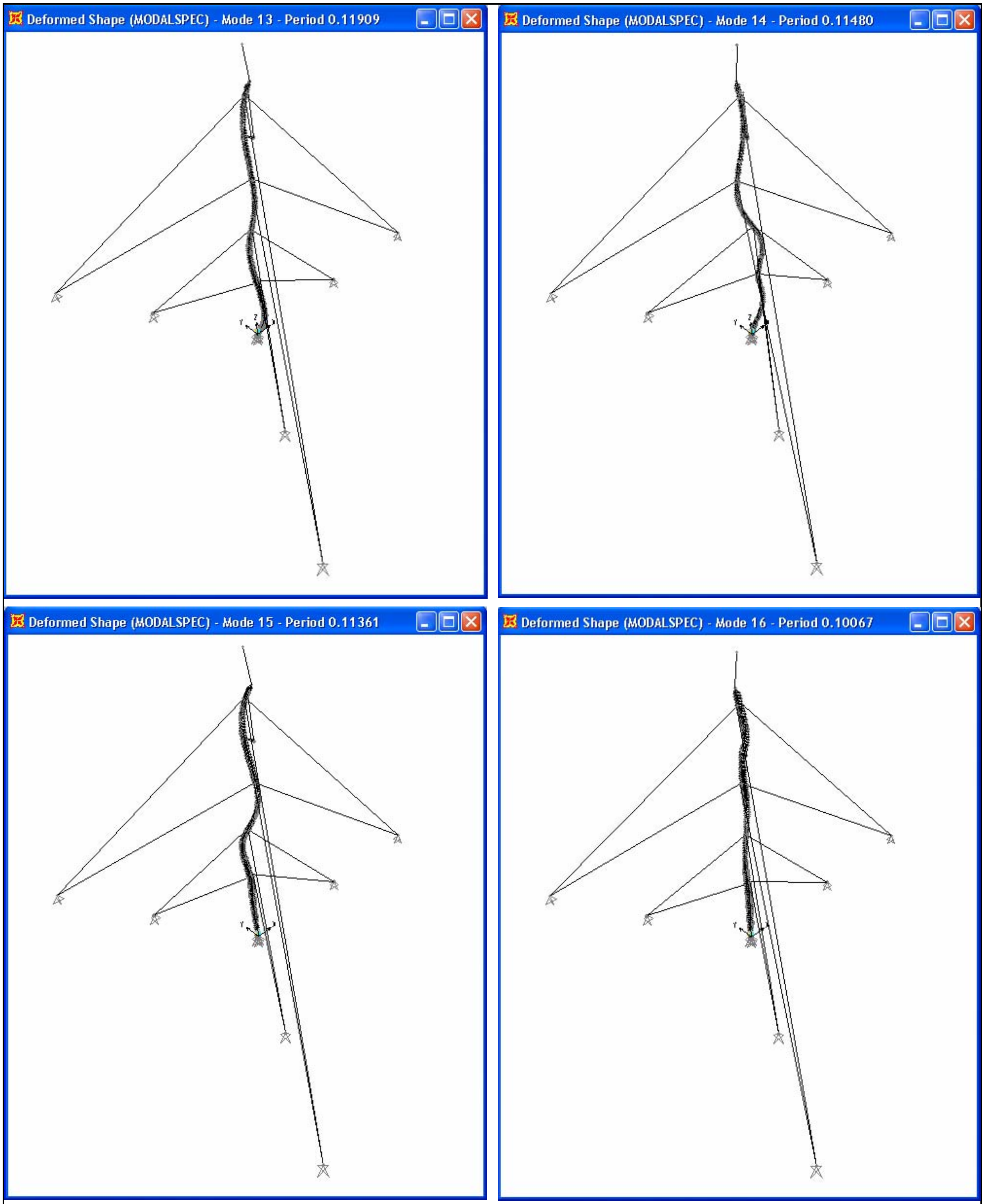
Mode shapes 1 - 4



Mode shapes 5 - 8

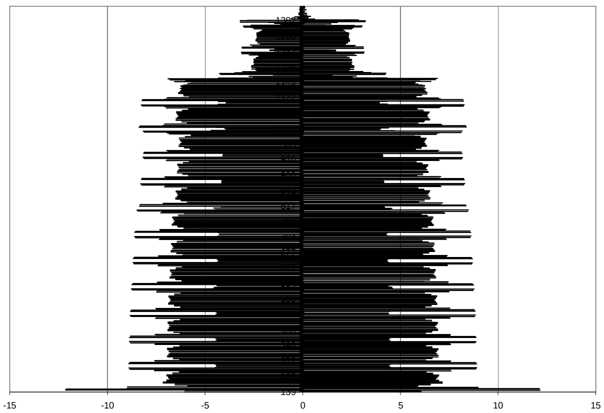


Mode shapes 9 - 12

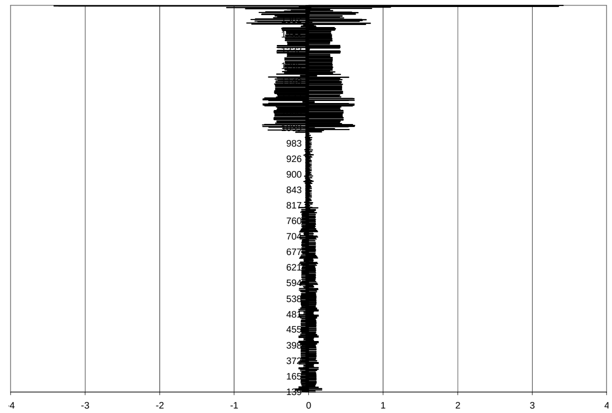


Mode shapes 13 - 16

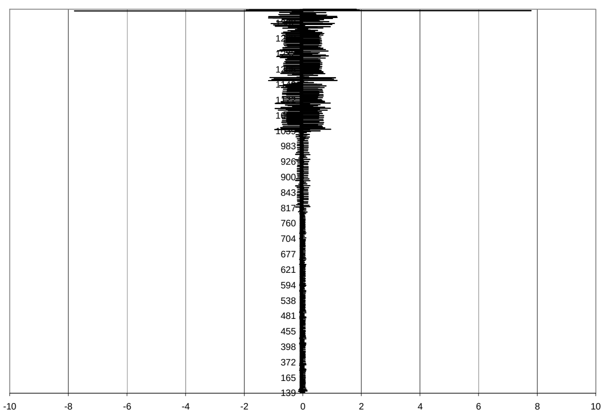
Axial Forces for Braces for All 16 Modes for Response Spectrum Function



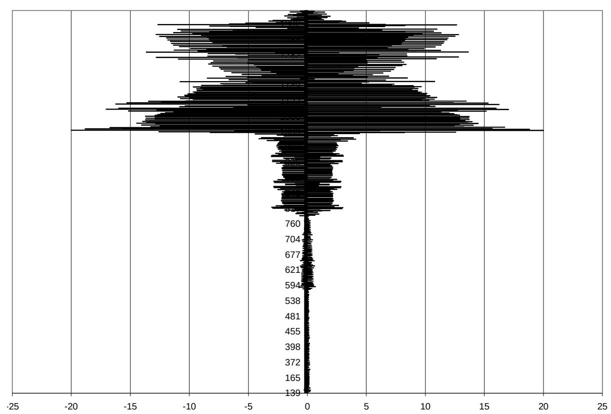
Mode 1



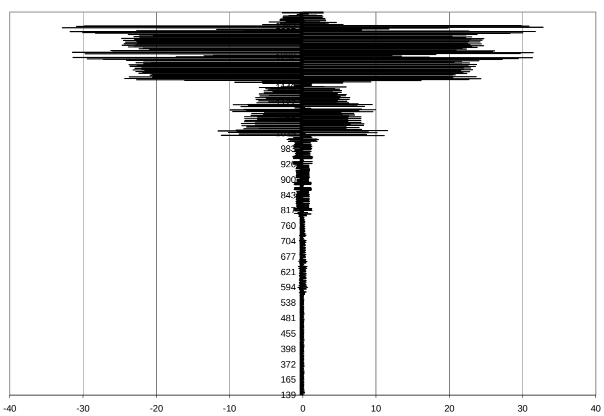
Mode 2



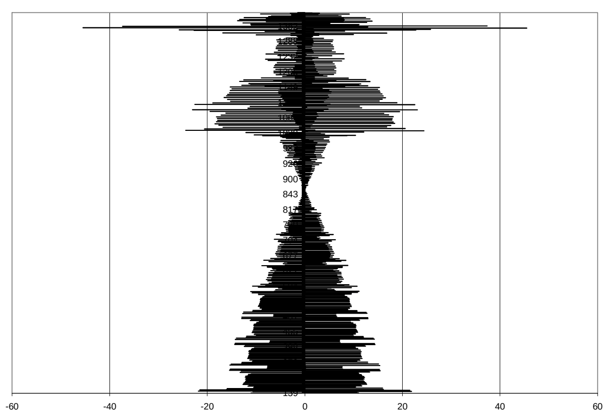
Mode 3



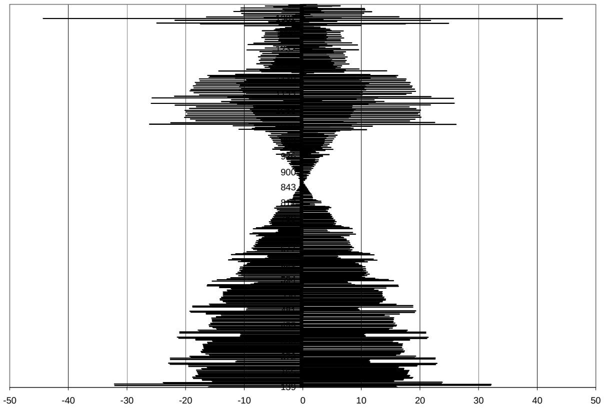
Mode 4



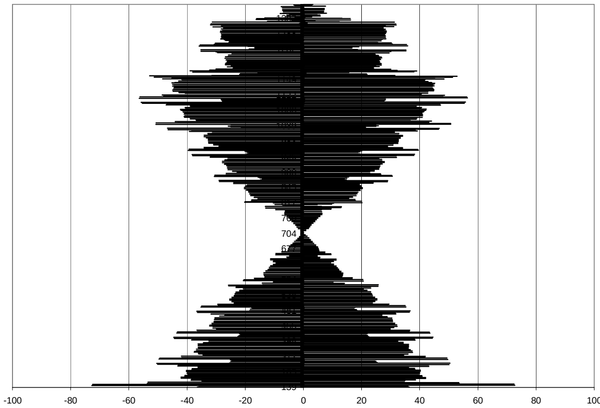
Mode 5



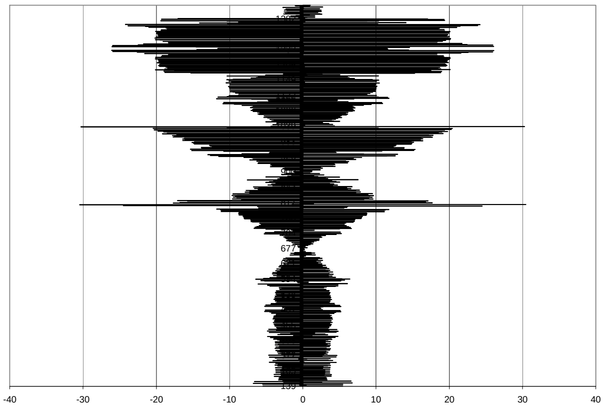
Mode 6



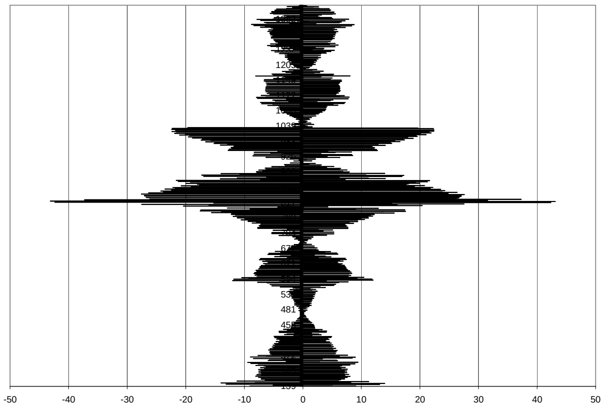
Mode 7



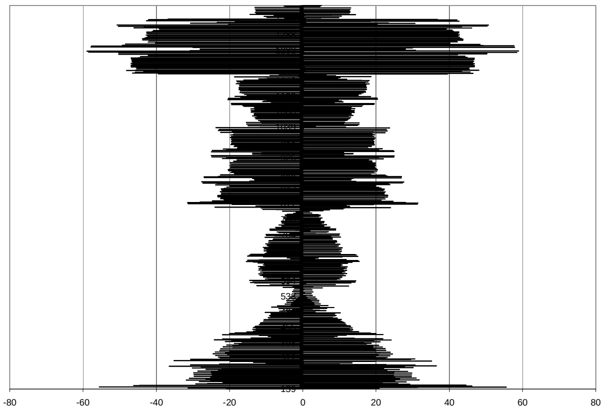
Mode 8



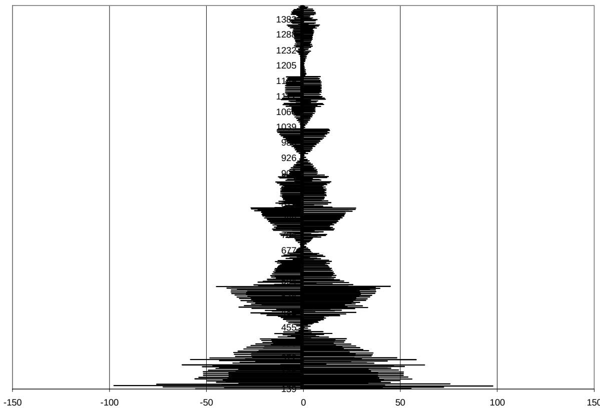
Mode 9



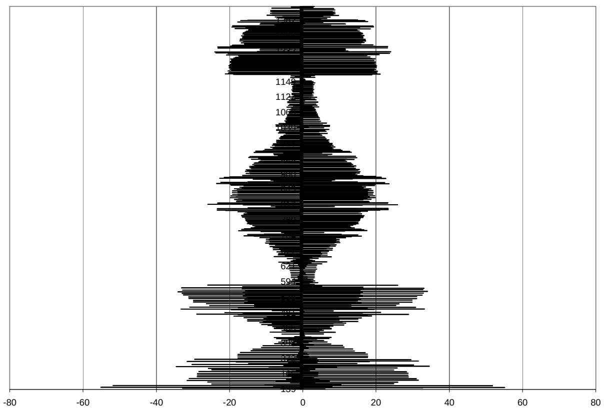
Mode 10



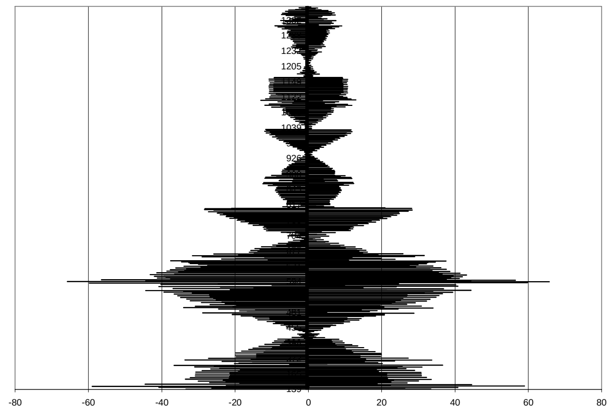
Mode 11



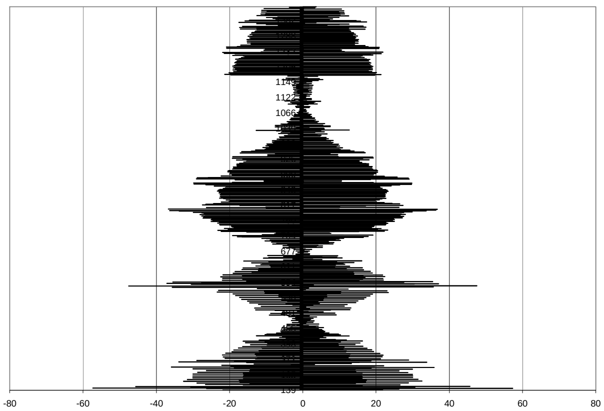
Mode 12



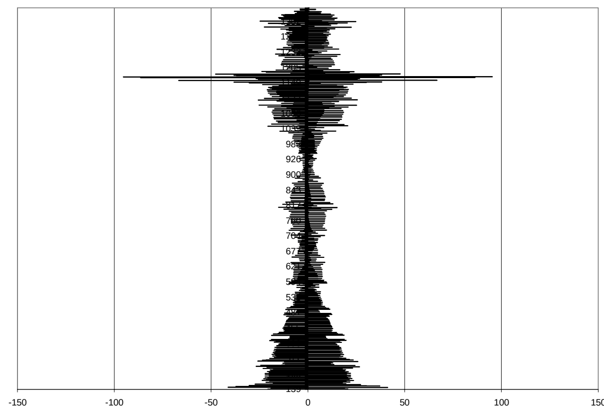
Mode 13



Mode 14



Mode 15



Mode 16

Appendix F. Select Tower Design, Analysis and Maintenance Consultants

Davidson Engineering
(Communications Structure Engineering)
296 Covered Bridge Road
Rogue River, OR
Phone: (541) 582-8074
Fax: (541) 582-0072
www.tower-structures.com/index.html

Sioux Falls Tower Specialists, Inc.
2224 East 39th Street North
Sioux Falls, SD 57104
Phone: (605) 331-6972
Fax (605) 332-7833
www.siouxfallstower.com

Intermountain Tower Specialists, Inc.
P.O. Box 1943
438 S. Commerce Rd
Orem, UT 84059
Phone: (801) 434-9883
Fax: (801) 434-9153
www.intermountaintower.com

Nationwide Tower Company
P.O. Box 1829
Henderson, KY 42419-1829
Phone: 270-869-8000
Fax: 270-869-8500
www.nationwidetower.com

SiteMaster Tower Maintenance and Construction Services
Phone: (918) 663-2232
Fax: (918) 663 – 2291
www.sitemaster.com