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Simplified seismic analysis methods for guyed telecommunication masts

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Preface

In the early 2007, Brian Smith, a long-time (now retired) member of IASS WG4 (International Association for Shell and Spatial Structures - Working Group Number 4 on Masts and Towers), published a remarkable book titled *Communication Structures* [1]. His book is deep, sharp and provides a thorough study of the technology of telecommunication structures. It reviews literature and worldwide experience in the analysis, design, construction, and operation of telecommunication structures, combining the accumulated knowledge of structural engineers, researchers, and mast and tower owners. According to Smith, he has received a great deal of help and advice from the members of WG4, as the backing group, which is much appreciated.

In the foreword of Smith's book, written by Y.B. Yang, the chair of the technical activities committee of IASS, it is mentioned that "The importance of communication structures cannot be underestimated, as the failure of communication facilities can sometimes be quite destructive. One example was the malfunction of the communication facilities of a key station in the Kobe area of Japan immediately after the January 17, 1995 earthquake. This event was said to have prevented local governments knowing the level and scope of casualties caused by the disaster; as we know, this is crucial for the rescue of injured people from collapsed buildings in the first few critical hours." Furthermore, in the context of the 18th chapter of this book, it is reported that there was some damage to communication towers during the devastating August 1999 earthquake in Turkey, although details have not been in the public domain.

These illuminating reports have extremely impressed me during the first months of my studies in McGill. I was seeking some justification for my belief that the seismic response of guyed telecommunication structures was worthy of special attention, considering their crucial role in post-disaster communication networks. As the topic of my PhD research, I deeply believe in the inherent significance of the preservation of the essential infrastructures, such as guyed telecommunication towers, in the event of a severe earthquake; I was frustrated when I several times heard or read that "Wind effects, and the combination of wind and ice effects, are more likely to govern the structural design of communication structures than are earthquake effects" or "Although there have been many reports of structures failing due to extreme wind and/or ice, there have been only been isolated in connection with earthquakes, none of which having been a direct threat to injury or life". Smith's book was not my first approach to the subject, but it was surely the best one, which profoundly extended my understanding of the topic. Following its references, I was fully persuaded that dynamics of guyed telecommunication towers, especially in the case of an earthquake, deserves deep attention.

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It is worthy to acknowledge that the IASS Working Group 4 on Masts and Towers is an animated technical working group of enthusiastic international experts of the field of Telecommunication Structures. After studying Smith's book, I was very proud to attend the 23rd meeting of the IASS WG4 in Montréal on 9-13 September 2007, as well as to make a presentation on "Response of Telecommunication Masts to Seismic Wave Propagation" in collaboration with Farzad Faridafshin (a former research visitor at McGill) and Professor McClure. This was a precious opportunity for me to get more acquainted to the problems of this industry and to develop my network of technical contacts.

Finally, I would like to express my sincere appreciation to Professor Ghyslaine McClure who provided me with the opportunity of continuing my studies at McGill under her supervision as well as her financial support. Her kind support, precise supervision, and friendly treatment are really highly appreciated.

Throughout this report, special attention will be paid to reviewing former research at McGill by Guevara (1993), Amiri (1997), Dietrich (1999), and Faridafshin (2006). Furthermore, relevant recent books, namely *Communication Structures* by Brian Smith (as mentioned before) and *Dynamic Response of Lattice Towers and Guyed masts* edited by Murty K.S. Madugula, also a member of IASS WG4, and North American Codes, ANSI/TIA/EIA-222-G and CSA S37-01 will be reviewed carefully. Finally, other dissertations and articles from the rest of the open engineering literature will be considered in this report.

S. Ali Ghafari Oskoei

1. Introduction

1.1. General

Worldwide, billions of people daily take advantage of accessing telecommunication and broadcast TV and Radio services, brought to their houses or work places by means of various structures supporting the telecommunication antennas. The communication industry has deeply influenced our life, and we are used to keeping communication with others or receiving information from radio and TV every day. However, very few of people are aware of the importance to ensure that telecommunication structures be properly designed, constructed, and maintained. Tall guyed telecommunication structures pose several engineering challenges due to their highly nonlinear behaviour, especially when they come to dynamic effects. Therefore, they are eligible to deeper attention and more concern.

As another crucial aspect, telecommunication structures are among the fundamental components of communication and post-disaster networks, and their preservation in the case of severe earthquake is essential.

There are several types of telecommunication structures, such as Lattice Self-Supporting Towers, Lattice Masts stayed by steel cables (guys), Self-Supporting Concrete Towers and Masts, and Pole-type Structures. The present research deals with the dynamic behaviour of tall Guyed Telecommunication Masts under seismic loads. Telecommunication masts, also called guyed towers, are typically tall (height above 180 m), slender structures, whose lateral resistance is provided by clusters of guy cables anchored to the ground at several support points. Their function is to support elevated antennas for telecommunication, radio and television broadcasting, and two-way radio systems (emergency response systems such as police and fire). Therefore, immediate serviceability or even continuous function of these first-aid-station infrastructures is of critically high priority in the case of a disaster in the seismic-prone regions of the world. Figure 1-1 illustrates some details of a 111.2 m mast owned by Hydro-Québec in St-Hyacinthe, Québec, Canada, as an example of telecommunication mast.

Reviewing the engineering literature on the seismic behaviour of telecommunication structures, it seems noteworthy that although there have been several reports of structural failures due to extreme wind and/or ice, there have only been isolated reports in connection with earthquakes. McClure (1999) [2] quoted a survey of the earthquake performance of communication structures which summarizes documented reports of 16 instances of structural damage related to seven important earthquakes in the past 50 years. It should be mentioned that damage is not being systematically reported for these structures as such information is most often kept confidential by tower owners. However,

Figure 1-1. Example of a 111.2 m guyed telecommunication mast owned by Hydro-Québec in St-Hyacinthe, Québec, Canada.

quite recently, a consensus has been raised in North America to document earthquake-resistance design guidelines specifically developed for communication structures. This increased level of awareness of the seismic risk has encouraged researchers of this field to establish simplified rational procedures to evaluate the reliability of tall telecommunication structures, which is also the main motivation of the present study.

Several technical challenges and interesting considerations are associated with the seismic behaviour of guyed telecommunication masts. Primarily, most masts with height ranging typically from 150 m to 300 m have their fundamental flexural frequencies within the sensitive range with respect to the frequency content of usual earthquake ground accelerations. However, seismic effects are not likely to govern the design in areas with moderate seismic risks. Moreover, as another important issue, the mast/guy interaction could potentially trigger significant seismic effects in guyed telecommunication masts. This may happen when the vertical ground motion is combined with the usual horizontal motion, provided that there is a frequency coincidence between the input dominant frequencies and the frequencies of the dominant, strongly-coupled cable and mast modes, if such modes exist.

The methodology used in this study is based on the modeling of the seismic input in terms of the prescribed components of ground displacements along the three orthogonal directions at each support and with appropriate lack of correlation. Past studies have suggested that earthquake effects on the masts appear to be significant only in the top cantilevered section of tall masts (when such a feature is present) and in the first span near the base. They have also revealed that dynamic amplifications in the guy tensions are more likely to be significant in the top and bottom levels of multilevel guyed masts; relatively slack cables with initial tension below about 5% of their ultimate tensile strength are especially vulnerable.

To date, to the best knowledge of the author, simplified and quasi-static models for seismic design of guyed masts have not been proposed in the literature, unlike for wind actions. In the engineering practice, such models and methods would be appreciated. As such, in the Canadian Standard CSA-S37, Table M.1, it is recommended to perform a detailed dynamic analysis for all masts of height above 150 m and located in high seismicity areas and for all masts where continuous serviceability is needed in moderately active areas. Structures taller than 300 m should be subjected to a detailed analysis where there is a risk of injury or loss of life in moderately active areas, including the effect of asynchronous motion at the mast base and stay anchorages. But the studies that form the basis of the CSA-S37 Appendix M have shown that “detailed nonlinear seismic analyses are far more complex than response spectrum analysis and not always necessary”. Nevertheless, calculation of the natural frequencies of masts in their initial configuration can help identify the seismic sensitivity of these structures and the potential interaction effects due to clustered frequencies. The expected contribution of the research that will follow this literature review is to establish a simplified rational procedure for seismic design of tall guyed telecommunication structures.

1.2. Report outline

The present report comprises the following chapters:

Chapter 1: “Introduction” presents an introduction to guyed telecommunication masts and the objective and the scope of the research that will follow this literature review.

Chapter 2: “Recent studies at McGill University” summarizes the main findings of relevant work done in the Department of Civil Engineering and Applied Mechanics, which forms the background for the proposed research.

Chapter 3: “A review of theoretical developments” reviews the theory of nonlinear dynamic analysis of guyed masts.

Chapter 4: “Design guidelines” reviews the North American Codes in the field of guyed telecommunication towers.

Chapter 5: “Review of other works” covers material presented in the open scientific literature: articles, dissertations, and scientific reports.

2. Recent studies at McGill University

2.1. Introduction

The following sections present the recent work on tall masts dynamics carried out in the department of Civil Engineering and Applied Mechanics of McGill by Guevara (1993-1994), Amiri (1993-1997), Dietrich (1999), and Faridafshin (2006), under the supervision of Professor McClure.

2.2. Guevara (1993-1994)

The nonlinear seismic response of three guyed telecommunication towers was first studied by Guevara [3] (also reported in Guevara and McClure (1993) [4] and McClure and Guevara (1994) [5]). It was an exploratory study using detailed nonlinear finite element analysis models of three masts only. In a review of this work in Madugula (Ed. 2002) [6], it was stated that Guevara's simulations raised more questions than they answered, namely in relation to cable-mast interaction, multiple-support excitation, and response to coupled vertical and horizontal inputs. On the other hand, the work was ground breaking and raised the level of awareness to the importance of dynamic effects on guyed masts.

In the literature review section of his study, Guevara stated that traditional attempts at the numerical modelling of guyed towers, for instance by McCraffrey and Hartman (1972), Augusti et al. (1986), and Ekhande and Madugula (1988), were accompanied by simplifications in the model, such as replacing the cables by equivalent springs or substituting the masts with equivalent Timoshenko beam-columns. Later on, however, studies by Augusti et al. (1990) and Argyris and Mlejnek (1991) revealed that although these simplifications are not significantly influential when studying the static response of structure, they are no longer appropriate for dynamic analyses of guyed masts.

One work reported by Augusti et al. (1990) included the modelling of a 200 m guyed mast with three guying levels, in which equivalent static linear springs were employed to model the guy cables and where the equivalent stiffness of the springs varied with the frequency of oscillation. However, the inertia effects of the cables were neglected. Furthermore, another numerical study by Raman et al. (1988) clearly confirmed the importance of geometric nonlinearities in the guyed tower responses under quasi-static loads. However, several aspects of dynamic analysis of guyed towers required improvements at the time when it was difficult to create precise structural models.

Recognizing the deficiencies in previous studies, Guevara's work concentrated on detailed numerical modelling of three guyed telecommunication towers in the time domain, namely a 24 m tower with two stay levels, a 107 m tower with six stay levels, and a 324 m tower with seven stay levels. The towers were subjected to S00E 1940 El Centro and N65E 1966 Parkfield accelerograms, which were scaled down to match the elastic design spectra of the 1990 National Building Code of Canada for the Montréal Region. Special attention was paid to the combination of horizontal and vertical ground accelerations in the case of the tallest mast. In addition, surface wave propagation was also studied by considering asynchronous inputs at the ground support points of the guyed tower using the LMM (Large Mass Method). The shaft of the first two masts were modeled as equivalent Timoshenko beam-columns, using the Timoshenko's Beam Theory and Saint-Venant's Torsional Theory to establish the corresponding shear and bending rigidities. The axial rigidity was directly obtained from the cross-sectional area of the legs. However, a detailed three-dimensional truss model was used for the tallest mast. Structural damping was not added but artificial numerical damping was provided by the direct time-step integration method. In addition, it was assumed – also for the rest of studies – that earthquakes occurred under still air conditions and aerodynamic damping was neglected and not modeled.

Guevara's study indicated that the high frequency component of the excitation affected only the shortest tower. More significant dynamic amplifications were found in the extreme guy clusters, i.e. top and bottom, for the response of the two other structures. In addition, due to the distortion caused by the unsymmetrical layout of diagonals and some other geometric simplifications, an error of less than 3% for the 24 m tower and less than 5% for the 150 m tower was induced in the equivalent model, in comparison with the 3-D model. The mast equivalent models were validated through a frequency analysis of the equilibrium configuration under the dead weight of the structure and the cable pretensioning force. There were discrepancies between the equivalent model and the detailed model in terms of the fourth and the fifth mode shapes near the base of the mast, which was stated to be related to refinements. But apart from local effects, the first five modes, and more importantly the corresponding frequencies, were reasonably well presented in the equivalent model. On the other hand, detailed modeling of the tallest mast was recommended, because of the large number of different member properties along the height of the mast. In addition, it was noted that correct modeling of the torsional behaviour of the tallest mast could not be achieved with the equivalent beam-column formulation.

Guevara also found that asynchronous ground acceleration had significant effects only in the guy wire tensions of the bottom cluster for the shorter masts. However, in the case of the tallest mast, multiple support excitations caused additional dynamic effects that were not presented when only synchronous ground motion was simulated. The work also indicated that cable-mast interactions were dominating in the frequency range of lower axial modes of the mast. Important dynamic interactions were found between the mast and the guy wires when horizontal and vertical ground accelerations were combined as input.

2.3. Ghodrati Amiri (1997)

2.3.1. General

With the main objective of proposing some seismic sensitivity indicators for the design of tall telecommunication masts, Amiri (1997) [7] studied eight existing structures, varying in height between 150 to 607 m. It was anticipated that assessing the sensitivity of a guyed mast to earthquake effects with simple indicators would be a first step to determine whether a detailed nonlinear dynamic study is necessary.

The shaft structures were modeled as three-dimensional trusses. An equivalent viscous damper with a value of 2% of critical viscous damping was used in parallel with each element to model structural damping. Three different classical seismic excitations (namely El Centro, Parkfield, and Taft) were applied as an acceleration-based input. In some simulations, the horizontal acceleration was accompanied by a synchronous vertical component with 75% of the horizontal amplitude.

A short review of his conclusions, which were later enriched in Amiri (2002) [8], is presented in the following section.

2.3.2. Natural frequencies and mode shapes of guyed telecommunication towers

Amiri devoted a considerable portion of his research on investigating the natural frequencies and mode shapes of guyed telecommunication towers. As such, it is indicated to present here a comprehensive coverage of the modal characteristics of these structures, including but not restricted to Amiri's results.

As with any structural dynamic problem, the response of a guyed telecommunication tower to a dynamic excitation will be affected by its natural frequencies and mode shapes; modes with natural frequencies that coincide with the frequency content of the input dominate the response. Moreover, as a common engineering practice, it has been shown that for most dynamic analyses under lateral loads, the accurate prediction of the lowest five flexural modes suffice for self-supporting towers. Nevertheless, guyed towers have no normal modes of vibration, as their stiffness is geometrically nonlinear. Despite seeming not to be quite rational to evaluate, their eigenproperties can be considered as an indicator of the dynamic sensitivity of the structure or even may be used in a linearized dynamic analysis. To this end, in order to evaluate the eigenproperties, a specific initial condition must be prescribed which must be compatible with the dynamic effects to be studied. For seismic analysis, for instance, it is recommended to evaluate the natural frequencies and the mode shapes of the initial static configuration of the structure under the self weight and the cable prestress. Unlike most conventional free-standing structures, guyed telecommunication towers routinely exhibit twenty or more active modes when excited by turbulent winds, in the frequency range below 0.3 Hz. Earthquakes are likely to excite modes of vibration in the range of 0.1-10 Hz, which may include several modes in tall masts.

In a section on the experimental measurements of the natural frequencies of telecommunication towers, Madugula Ed. (2002) [6] stated that, it was relatively simple, using the spectral analysis of digitized data, to extract the dominant frequencies of a recorded response indicator following a dynamic excitation or even under ambient vibrations. The collection of experimental data on mode shapes, however, was not straightforward, unless in the presence of near-resonance conditions and well-separated modes. As such, this exercise is particularly difficult for masts, since guy cables often experience multiple resonances.

However, in the case of self-supporting towers, it is common practice to consider linear dynamic behavior, which implies the existence of normal modes of vibration. According to Khedr (1998) [9], good separation of the flexural frequencies was usually observed, while the torsional and the flexural modes were often nearly coupled; the axial modes were always well separated from lateral modes. Therefore, he concluded that it would be acceptable to assume uncoupled behavior for the vertical and horizontal directions in the case of self-supporting towers. Galvez (1995) [10,11] developed a simplified procedure to assess the fundamental frequencies of the self-supporting lattice towers, considering that self-supporting towers behave essentially as cantilever beams. Further, based on simple prismatic cantilever beam theory, Sackmann (1996) [12] suggested simple empirical improvements to the predictions by Galvez. Nowadays, guidelines on the eigenproperties of self-supporting towers can be found in some National Codes, for instance in the European Code [13] or Appendix D of the Australian Standard AS 3995-1994 “Design of steel lattice towers and masts” [14], which can be helpful for designers in this field.

When considering the modes and frequencies of guyed towers, two sets of properties are worthy of attention: first, those of the guy wires alone, in order to identify potential adverse localized vibrations and fatigue, and second, those of the overall mast. The usual practice is to refer to the fundamental frequency of a mast as the one corresponding to the fundamental transverse mode of the longest guy cables, connected to the top of the mast. Unlike self-supporting structures, guyed masts have several closely-spaced lateral and flexural modes, with the first 15 modes frequently below 3 Hz.

The dominant mode shapes of a guyed mast are strongly influenced by the guying configuration and the relative lateral stiffness of the various guying levels. The mast is usually pinned at the base, but the presence of stiff guy wires near the base may provide enough lateral stiffness to mimic a nearly fixed-end behavior. At the other extreme, the long guy wires attached near the top of very tall towers are very flexible and provide little lateral stiffness, such that in some cases the top of the mast may behave almost as a free cantilever. Just in the case of the first lateral mode shape, it could be seen that the top guys provide enough lateral resistance to approach a pinned condition.

Amiri (1997) [7] proposed an empirical expression to evaluate the fundamental flexural period of the mast (in sec) as follows:

$$T = 0.0083L - 0.74 \qquad 2.1$$

Equation 2.1 was derived for bare steel lattice masts of triangular cross sections with heights ranging from 150 to 607 m with the guy wire initial tensions in the range of 10% of their ultimate tensile strength. However, due to lack of data for towers above 350 m, it is cautioned to limit the use of this formula to the 150-350 m range. Wahba (1999) [15] proposed another empirical equation, compatible with Amiri's. His study further confirmed that the total height was the most direct factor in determining the lowest natural frequency of guyed towers. He concluded that rigorous dynamic analyses were not warranted for guyed towers less than 200 m in height.

Considering these efforts, it is fair to argue that empirical equations which estimate the fundamental frequency of a guyed mast as a function of a unique parameter, such as height, cannot be reliable in the worldwide scale of use. It may be justified to extract such empirical equations for the towers of a common category. However, a very large number of real towers need to be analyzed to obtain reasonable confidence level. From a more scientific point of view, the fundamental frequency of structures is a function of the stiffness and the mass of structure, involving several key parameters, for instance the number and the configuration of the guy wires, their cross-sectional area and level of the initial tension, and the distribution of the reactive mass of structure along the height. Nevertheless, none of these essential parameters are considered in the previous studies.

On the other hand, Appendix D of the Australian Standard [14] suggests expression 2.2, in order to estimate the fundamental natural frequency of guyed masts in a direction parallel to the design wind speed.

$$f_1 = 0.15\sqrt{\frac{(k_1 + k_N)(N+1)}{N\left[M_T + \sum_{j=1}^{N_A} 5M_{Aj}\left(\frac{y_i}{h}\right)^4\right]}} \qquad 2.2$$

Where

k_1, k_N: Minimum lateral stiffness for the first (bottom) and last (top) cluster systems on the mast for any wind direction

N : Number of guy levels

M_T: Total mass of the mast section and half of the total mass of all guy clusters attached to it

N_A: Number of ancillaries (antennas or other heavy attachments)

M_A: Individual mass of ancillaries

y_j : Height of ancillaries above the ground

h: Height of the mass above the ground

This expression should yield lower frequencies than in still air conditions. It was also derived for towers of limited heights (below 200). Although the empirical formulas derived by Amiri or Wahba can be used in very specific situations (within the limits of the data used in their derivation) for very crude estimates, Equation 2.2 appears more rational as it tries to consider the key parameters in terms of stiffness and mass distribution.

2.3.3. Seismic response indicators of guyed telecommunication towers

The conclusions drawn from Amiri's study were employed to propose some simplified models. In general, the trend of most response indicators - including base shear, axial force in the mast, cable tension, shear force and bending moment of the mast, and deformations - was to increase with tower elevation. There was, however, a discontinuity in the trend of the tower lateral stiffness in the transitional portion from the inner or the outer anchor points. This area was a sensitive portion of the tower where most of the response indicators showed a nonuniform behaviour. With exception of the dynamic component of the mast axial force and the mast rotation, the maximum dynamic amplitudes of the other indicators, such as shear force and bending moment of the mast, occurred close to the transition zone. A summary of the main observations is presented in the following paragraphs.

Total shear force

The total shear force for a guyed tower was defined as the sum of the horizontal reactions at the base of the mast, including the effects of the guy wires at the ground anchor points. For tower heights ranging from 150 m to 350 m, important base shear may develop in the order of 40% to 80% of tower weight, depending on the seismicity of the area. The magnitude of the base shear, for peak ground accelerations of 0.34 g (specified for Victoria in British Columbia, Canada in the 1995 NBC [16]), could be predicted using the following equation [8].

$$B.S. = 28\,300\ H^{-1.17} \qquad 2.3$$

where $B.S.$ is the maximum base shear as a percentage of total weight W, and H is the tower height in meter. This applies to towers on rigid base where no soil-structure interaction is considered. For towers taller than 350 m, the relative contributions of the inertia effects in the cables and the mast were unpredictable, and it was not possible to suggest a simple estimator in these cases; a detailed dynamic analysis was therefore recommended. This statement emphasizes the need to better understand the physics of the cable mast interaction, and according to author, a simple model of continuous beam resting on flexible supports is worthy of attention. It should also be noted that the definition of seismicity of the Victoria region has been changed in the NBC 2005, so Eq. 2.3 is no longer valid.

Axial compression in the mast

The axial force in the mast is another important response indicator that was studied in some depth by Amiri. As it had been noticed before by Guevara (1994), guyed towers with slack cables were sensitive to the combination of the vertical and the horizontal earthquake motions. Since their behavior was unpredictable, a detailed nonlinear dynamic analysis was recommended. For guyed towers with usual cable pre-tensioning, i.e. around 10% UTS (Ultimate Tensile Strength) or more, the maximum dynamic component of the axial force at the base of the mast due to the combined vertical and horizontal earthquake motion was about 80% of its total weight, for the Victoria region seismicity level discussed above. Nonetheless, a detailed numerical study was not deemed necessary unless there was information available on the vertical accelerograms which would indicate that their frequency content was much different than that of the horizontal accelerograms. It could be seen that the results of the distribution of the axial force along the mast were well correlated (Amiri 1997) [7], which seems to be in contrast with his former statement on unpredictability of the behavior of the masts. The axial force profile could conservatively be represented by a parabolic curve fitted with the following equation:

$$\left(P_{dyn}/BA\right)=100-95\left(h/H\right)^2 \quad (in\ \%) \qquad 2.4$$

where $\left(P_{dyn}/BA\right)$: is the percentage ratio of the maximum dynamic component of axial force in the mast at a section of elevation h to the maximum dynamic component of the axial force at the base of the mast.

Horizontal shear force and bending moment in the mast

Results also indicated that, in the lower range of tower heights (150 – 213 m), the maximum shear varied around 6–7% of the tower weight, and the ratio of the maximum bending moment to the product of the panel width and the total weight of tower varied from 36% to 48%. In the upper range of tower heights (313 – 607 m), the effects were slightly smaller in percentage, but not in absolute values, with the maximum shear in the range of 2.5 to 5% of the tower weight and the corresponding bending moment ratio in the range of 14 to 35%. It was confirmed that the maximum values of mast shear occurred directly at stay levels and the minimum shear occurred at the mid span between two stay levels, and vice versa for the mast bending moment.

Distribution of earthquake force along tower height

The earthquake forces, generated by the inertia effects, affect both the mast and the cables of a guyed mast. The predominant mode shape of the mast could be used to represent the horizontal acceleration profile along the tower height. This acceleration profile combined to the mass profile could represent the distribution of the horizontal earthquake forces developed in the total structure. Moreover, a simplified conceptual model was proposed to explain the earthquake force distribution along tower height, which made use of three important tower characteristics:

1- The predominant mode shape of mast.
2- The structure mass distribution.
3- The presence of discontinuities in lateral stiffness.

However, for most towers, the maximum lateral force on the mast exerted by the cables varied in the range of 25 to 35% of the total base shear. Owing to cable/mast interactions, the distribution of the lateral force on the mast could not be explained only by the mass profile of the mast and its fundamental mode shape, as is the case for lateral effects in regular lightly damped buildings. The non uniform distribution of the lateral stiffness provided by the guying system plays an important role. However, it is worthy of note that the mast accounted for 69% to 77% of the total tower weight, and leaving 23% to 31% to the guy cables. It could be seen that the variation of the mass of the tower along the height of the tower was almost uniform, except for stabilizer locations, and the mass per unit length of the towers studied varied from 141 kg/m for the 150 m tower to 582 kg/m for the 607 m tower.

As another interesting point, the dynamic component of the cable tensions varied by one order of magnitude, from 30% to 300% of the initial tension for the towers studied. Typical values for the 150 to 350 m range of tower height were between 50% and 200%.

Finally, one of the strangest statements in Amiri's conclusions was that the inertia effects in the guy cables contributed little to lateral seismic effects in the mast and could be neglected. However, a careful review of other sources of information in the engineering literature contradicts this statement. The role of the inertia effects of the cables in generating cable-mast interactions for realistic three-component of ground motion has been confirmed in the recent work of Faridafshin (2006). However, the complexity of these phenomena still calls for more in-depth studies.

Amiri further argued that when vertical accelerations were studied, dynamic interactions occurred between the cables and the mast. These interactions were difficult to predict, and detailed analysis was recommended for very tall towers above 350 m or towers with relatively slack guy clusters, which highlights that empirical equations are inappropriate to predict several aspects of the behaviour of tall masts.

Tower displacements and rotations

Lastly, in the case of serviceability criteria, the lateral displacements in the earthquake direction were small, in the range of 0.05–0.12% of the tower height, which confirmed that, in spite of being slender in their appearance, the towers were not very flexible. The maximum flexural rotation (tilting) of the top of the mast was also below 0.4 degree for each tower, which should meet the serviceability of most reflector antennae. Consequently, the maximum dynamic component of cable oscillation varied between 0.3 and 1.3 m, which were small compared to the corresponding tower heights, and the dynamic component of the maximum axial displacement of the mast was found negligible in most towers. In general, then, for the seismicity level considered in the study, all the towers appeared to meet reasonable serviceability limits.

2.4. Dietrich (1999)

Dietrich's study (1999) [17] was as a follow-up study on Amiri's work although it was much more limited in scope: only one 150m mast was analysed but in a lot of details exploring the effects of the space and time variability of earthquake ground motions.

A typical analysis of a given structure takes into account the variation in time of ground motions, but ignores their spatial variability. The effect of spatial vibration of earthquake ground motion on the dynamic response of multiple-support structures may be important. The foundation dimensions of large structures such as guyed telecommunication towers are comparable to the wave length of the earthquake ground motion. Since the speed with which the pulse from an earthquake travels is finite, the assumption that every point at the base of these structures experiences the same acceleration at any instant is clearly inaccurate. It is also generally recognized that in multiple-support systems, such as guyed masts, each support might be excited differently than the others due to the distance between supports and the differences in the geologic and topographic features at their locations, namely site effects.

Until the late 1990s, the modeling procedures in commercial software did not allow to simulate three-dimensional asynchronous shaking realistically. As a result, indirect and penalty techniques were extended to compensate the simplifying modeling assumptions of synchronous ground motions. A study of this issue has been done by Léger et al. (1990) [18]. They stated that, previously, two simple deterministic analytical techniques, based on the traveling wave assumption, were used to take into account the spatial variation of the input ground motion, namely the Relative Motion Method (RMM) and the Large Mass Method (LMM). The RMM method is based on the principle of superposition. However, for large structures, it proves very time consuming since it requires an additional static solution at each time step used for the integration of the dynamic equilibrium equations. Also, the RMM method could not directly be extended to the study the nonlinear response of multiple-support structures.

Léger et al. further continued that, on the other hand, fictitious large mass values were attributed at each driven nodal degree-of-freedom in the LMM method. A critical modeling parameter for the LMM method is the value of the large mass used at each driven nodal degree-of-freedom. The LMM did not originate from a precise mathematical formulation, unlike the RMM, and the existence of the large masses in some models could cause numerical difficulties during modal extraction. Therefore, for a reliable implementation of this method, numerical sensitivity analyses were recommended. Nevertheless, the LMM could be applied directly in a step-by-step integration procedure carried out in geometric coordinates to compute the nonlinear earthquake response of multi-support structures.

However, these methods were accompanied with critical theoretical and numerical limitations. Recently, these limitations have been overcome, and now, it is feasible to achieve much more realistic computational simulations of tall masts under seismic loading. ADINA, Automatic Dynamic Incremental Nonlinear Analysis, finite element software for structures analysis, heat transfer and CFD problem, was employed by Dietrich (1999). The new version of ADINA was enhanced with improvements in the modelling of displacement-controlled ground motions through assigning time delays in each support point. It was also capable of modeling the vertical component of the ground motions simultaneously. Dietrich's mast model was the first that specified actual realistic three-component ground displacements as input. A comparison of these results with those generated by Amiri for the same mast indicated some shortcomings of the former approach. In particular, the effects of vertical ground motion and out-of-plane response to horizontal input were studied in detail. His study also modeled directly the effects of asynchronous ground motion and the presence of ancillary components.

2.5. Faridafshin (2006)

2.5.1. General

The main objective of Faridafshin's study (2006) [19] was to clarify some of the previous results obtained by Amiri (1997) and Dietrich (1999) and attempt to identify more definite trends in the calculated response of tall masts subjected to realistic three-dimensional ground motion following the approach tested by Dietrich with ADINA.

In Faridafshin's study, three existing masts with heights of 213, 313, and 607 m and different guy cable arrangements were modelled and investigated in detail. The geometric characteristics of the three masts are summarized in Table 2-1, and a schematic of the 607 m mast is illustrated in Figure 2-1, to provide an example. It is remarkable that the 607 m mast is among the tallest man-made structures and is located in Sacramento, California.

The first step, as in previous studies, was to study the eigenproperties of the models, mainly in order to verify them in comparison with Amiri's models. Furthermore, following a series of simulations modelling the earthquake excitation as synchronous shaking as the reference values, the effect of vertical component and the asynchronous input were investigated, assuming the tower on different site conditions with various surface shear wave velocities [20]. The main new contribution of Faridafshin was the consideration of various soil conditions under realistic shaking (asynchronous input), essentially affecting the time delays between the various ground support points. Moreover, a special

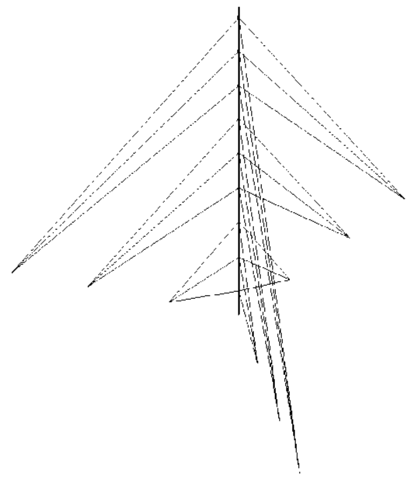

Figure 2-1. Geometry of the 607 m mast by Faridafshin (2006) [19].

section was devoted to the study of modeling the structural damping by parallel equivalent viscous dashpots. More information associated with structural modeling can be found in Faridafshin and McClure (2008) [21].

2.5.2. Seismic wave propagation effects on telecommunication masts

It is worthy of note that several efforts were devoted to studying the effects of the spatial variation of ground motions in the seismic analysis of various types of multi-support structures. The effect of asynchronous ground shaking was studied

Table 2-1. Geometry of the three masts studied by Faridafshin (2006) [19].

Height (m)	No. of stay levels	No. of anchor groups	Panel width (m)	Panel Height (m)	Location
607.1	9	3	3	2.25	U.S.A, California, Sacramento, (LeBLANC & Royle Telecom Inc.)
313.9	5	2	2.14	1.52	Canada (Wahba et al. 1992)
213.4	7	2	1.52	1.52	Canada (Wahba et al. 1992)

by O'Rourke and Hmadit (1988) [22], O'Rourke et al. (1980) [23], Dumanoglu and Severn (1989) [24], Lai (1983) [25], and Haroun and Abdel-Hafez (1987) [26] for pipelines, long multi-span bridges, and large dams; all of them confirmed the importance of considering the effect of multi-support ground excitations in the structural analyses.

However, traditional sources of information, such as seismological and geomechanics models, have been poor and unreliable in the course of producing data on spatial variation of earthquakes at the scale of engineering structures; several questionable assumptions were usually inevitable to compensate the lack of information and knowledge. Significant advances in the measurement and analysis of differential seismic ground motion have recently been obtained through the employment of arrays of strong ground motion accelerometers where a common time base allows the phases of the seismic waves to be correlated between recording elements. From the study of these records, it was confirmed that spatial correlations do exist as seismic waves propagate across the array site. Further studies suggested that in the case of multi-support structures, it is reasonable to assume only a phase lag between ground attachment points and ignore the change in the general shape of the signature where there is no local fracture or landslide potential.

The importance of multi-support seismic excitation of tall masts was also confirmed in a later study by Amiri et al. (2004) [27] through the investigation of the seismic response of a 607 m mast and a 342 m mast which had been studied previously. In this study, the TAB-TR component (horizontal component) of TABAS earthquake was employed. The main conclusion of this study was that by decreasing the shear wave velocity, or increasing the time delay, the maximum displacement of the tower tip decreased, mainly due to the effect of symmetric modes. The maximum base shear of the mast as well as the maximum axial force increased (In the overall order of 38% and 20% for 607 m and 342 m towers, respectively). The maximum tension of the inner cables increased considerably while this effect was less important or negligible in outer cables. However, the last two conclusions by the authors, namely the effect of symmetric modes on the maximum displacement of the tip of the mast and the negligible tension effects in outer cables, don't seem reliable to be extrapolated to other cases, and the study does not provide physical insight to the phenomena. Finally, as it was expected, the maximum horizontal component of the support reaction at cable attachment points decreases (In the order of 29% and 38% for 607 m and 342 m towers, respectively, in comparison with the synchronous input). Faridafshin (2006) [19] has improved the research in this field in several aspects. A comprehensive review of his work will be presented in the following sections.

2.5.3. Guyed tower models

In Faridafshin's study, the guy cables were modeled as straight lines in the initial undeformed configuration. Further, due to the static self-weight load and the pre-tensioning, they would deflect to their curved shapes, defined as the hyperbolic cosine profile. Consequently, the seismic loads were applied to the deformed geometry of the structure. The structure was modeled as pinned at the mast base. In those models in which the earthquakes were defined as accelerations, fixities were applied directly at the base. However, in the models with the displacement-based ground motions, zero displacements and free rotations (except twisting) were enforced at the base of the structure to model the boundary conditions in the static analysis.

The steel mast elements were assumed linear elastic with $E = 2.0E11\ N/m^2$, $\nu = 0.3$ and $\rho = 7850\ kg/m^3$, and the element type was the two-node truss. Large displacements and small strains were considered. According to Gantes et al. (1993), though the use of beam elements with semi-rigid connections created more accurate results, the more traditional way of using simple truss elements proved sufficiently accurate.

The dominant considerations in the case of the cable modeling were the use of a nonlinear tension-only material with the properties defined as $E = 1.73E11\ N/m^2$, $\nu = 0.3$ and $\rho = 7850\ kg/m^3$, and the three-node element was selected. Guevara and McClure [4] had conducted a convergence analysis on guy cable meshes and found that the parabolic (three-node) elements proved a good compromise in terms of accuracy and numerical effort. They also found that 10 to 35 tension-only elements per guy cable were required to model its five lowest frequency transverse modes of vibration. Gantes et al. (1993) indicated that 10 straight elements were sufficient and Faridafshin has used only 10 to decrease model size.

To model structural damping, all the mast and cable elements were supplemented by a parallel viscous dashpot damper with an equivalent of 2% critical viscous damping; however, some researchers argue that structural damping is smaller (in the range of 0.5-1.2%).

2.5.4. Numerical methods

According to Wilson (2000) [28], single-step, implicit, unconditionally stable numerical methods are recommended for time-domain seismic analysis of structures. Among the eligible methods, Faridafshin used the Newmark scheme, also known as the constant-average-acceleration method, with $\alpha = 0.5$ and

$\beta = 0.25$. For accuracy, Δt should satisfy $\omega_{co}\ \Delta t \leq 0.20$, where ω_{co} is the highest frequency of interest in dynamic response; $\Delta t = 0.001\ s$ was selected here. It was found that the BFGS Matrix Update Method is an effective scheme in models with large nonlinearities. Therefore, the BFGS method and the energy based convergence criteria were employed for simulations.

It is worthy of consideration that explicit methods are seldom employed in seismic problems. In contrast with implicit methods, which require solution of matrix equations during each time step, explicit ones involved no matrix equations and hence are simpler. However, the accuracy and the stability of direct time integration algorithms tend to be troublesome. As they are conditionally stable, explicit methods require small time steps for stability, often smaller than those necessary for the accuracy considerations. Conversely, implicit schemes are usually unconditionally stable, and the required time steps are usually larger than the one in explicit schemes.

The Wilson-theta method, obtained with a simple modification of the Newmark method is an unconditionally stable method (1973); however the θ factor tends to numerically damp out the higher modes of the system. Therefore, for problems where the high mode response is important, the errors introduced could be large. In addition, the dynamic equilibrium equations are not exactly satisfied at time t. At the time of introduction of the method, it solved problems associated with stability of the Newmark-family of methods. However, during the past 25 years new and more accurate numerical methods were developed and this one is no longer in use.

2.5.5. Post-Processing

In order to limit the volume of the output files, the nodes and time intervals might have to be limited. So, separate batch files (called *.plo* files in ADINA) were developed to deal with the response indicators of interest, such as the cable reaction force on mast, the cable tensions (3 points), the axial force, the bending moment, the shear force, the torsional moment all along the masts, the support reactions, the horizontal displacement and the tilt of the mast.

2.5.6. Modeling synchronous ground motion

As a reference for the rest of the study, the structures were analysed under synchronous ground excitations in the first step. This section of Faridafshin's study was a confirmation of the results obtained previously (by Amiri and Dietrich) and served as a benchmark for subsequent analysis. His findings in this section were almost the same as Amiri's, and verified the modeling procedure.

2.5.7. Modeling asynchronous ground motion

Faridafshin, Ghafari-Oskoei, and McClure (2007) [20] stated that, according to the principles of soil dynamics, there were three general causes for changes in the shape of the surface wave as it traversed the supports of a structure. First of all, if the material properties of the soil along the propagation path exhibited large variations between supports, it was expected that the wave shapes at the two stations would also differ. Secondly, localized wave reflections and refractions could cause changes in the wave shape. Thirdly, various types of seismic waves traveled at different speeds, and the time lag between two stations was not the same for different components of the seismic excitation, leading to changes in the wave shape. Seismic waves also attenuated as they propagated away from their source. However, for telecommunication masts, the concern was not the change in the shape of the surface wave because the footprint of these structures was not large enough to trigger significant variations in wave velocities or attenuation effects. It was, therefore, reasonable to assume that the spatial variation of the seismic excitation could be modeled as a travelling wave having a signature which remained unchanged as it traverses the structure.

They also continued that the effective velocity of seismic waves was of the same order as the shear wave velocity of the underlying soil. Thus, the wave travel time from one support point to the next was simply calculated from the ratio of the distance between the supports and the shear wave velocity. This is illustrated schematically on Figure 2-2 for the 607 m mast.

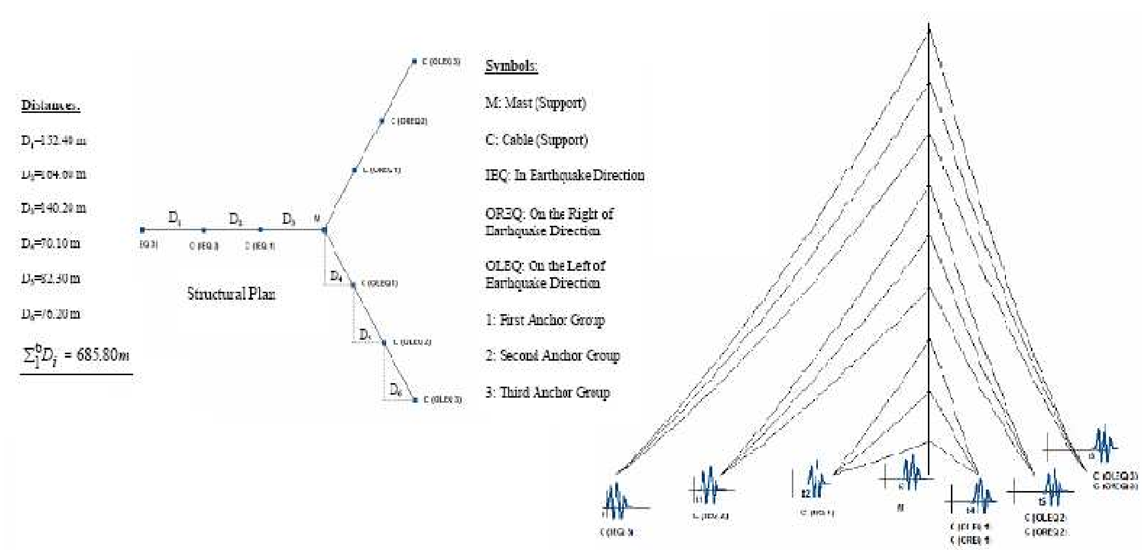

Figure 2-2. Schematic asynchronous ground motion input at support points for the 607 m mast [20].

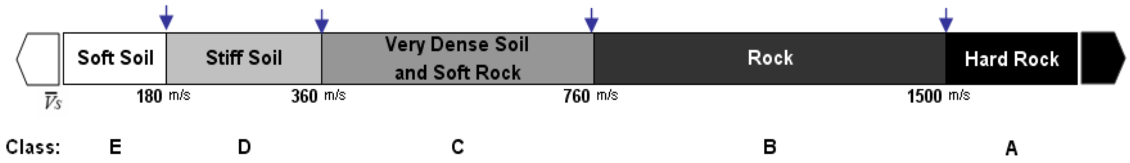

Figure 2-3. Soil categories and shear wave velocities, NBCC 2005 [16].

In Faridafshin's study, the site conditions (soil types) with their corresponding shear wave velocities were based on the classification of the National Building Code of Canada NBCC 2005 [16], which are represented schematically inFigure 2-3. The four boundaries between the site classes defined in Figure 2-3 (i.e. $\overline{V}_S$ = 180, 360, 760, 1500 m/s) were first considered to determine the sensitivity range for each tower. Then these intervals were successively reduced until the threshold value of shear wave velocity was identified.

2.5.8. Results and Discussion

Figure 2-4. Sensitivity to asynchronous shaking in relation to the shear wave velocity of the soil. [21] is a schematic summary of the results of all the simulations for the three towers by Faridafshin and McClure (2007) [21]. The arrows on the figure show the discrete velocities selected based on a bi-sectioning algorithm between the boundaries and taking into account the sensitivity and accuracy considerations.

Table 2-2. Geometry of the three masts studied by Faridafshin (2006) [19].

Earthquake	date	Magnitude (M)			Station	Site condition (USGS)
		M	MI	Ms		
Imperial Valley (El Centro)	5/19/194	7.0	-	7.2	117 El Centro Array #9	(C)
Kern County (Taft)	7/12/1952	7.4	-	7.7	1095 Taft Lincoln School	(B)
Parkfield	6/28/1966	6.1	6.1	-	1014 Cholame #5	(C)

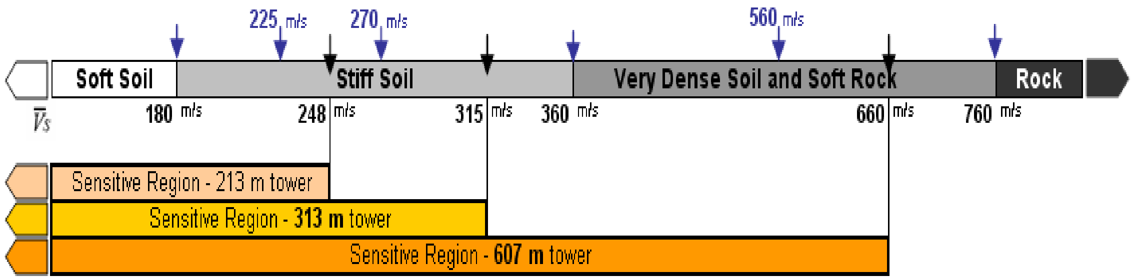

Figure 2-4. Sensitivity to asynchronous shaking in relation to the shear wave velocity of the soil. [21]

For the 213 m and 313 m masts, the influence of asynchronous shaking initiated on stiff soil conditions, starting with 248 and 315 m/s of shear wave velocities respectively. For the 607 m mast, however, the effect was significant in a much larger range of soil conditions, starting with 660 m/s of shear wave velocities which corresponds to very dense soil and soft rock, even toward the boundary for rock. For the tallest mast, a similar trend was observed in almost all response indicators. When the shear wave velocity and consequently the time lag between the support excitations increased, greater response was obtained in the structure. The mildest case was ST660, (e.g. ST660 corresponds to the soil type with shear wave velocity of 660 m/s) while the most severe case was ST180 which represented the boundary of soft soil. When the structure was founded on soft soil, which is usually avoided, very severe response might result for most of the response indicators. Figure 2-5, and Figure 2-6 present the envelopes of the mast shear distribution and of the bending moment distribution in X and Y directions caused by El Centro earthquake on the 607 m mast.

Different earthquake records with diverse scenarios of motion may produce quite different responses in the structures and the use of several records appropriate to the seismicity of the tower site is therefore necessary. However, the general trend was to have a larger response when moving toward a softer soil. Nevertheless, exceptions might exist at some guying levels as shown for instance on Figure 2-6 for the (X) component of bending moment on the mast between the elevations 100 m and 200 m.

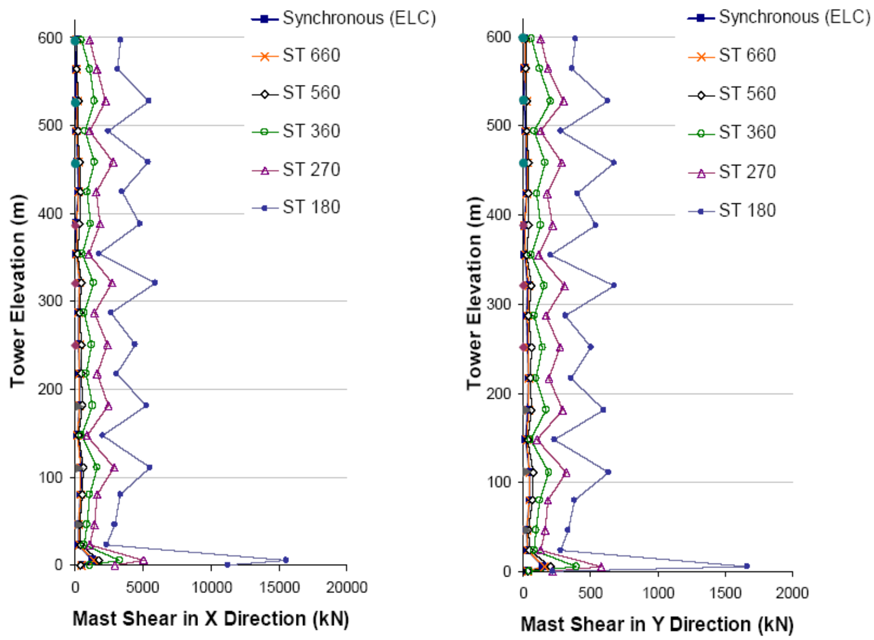

Figure 2-5. Envelope of the mast shear distribution in X and Y directions; El Centro; 607 m mast. [19]

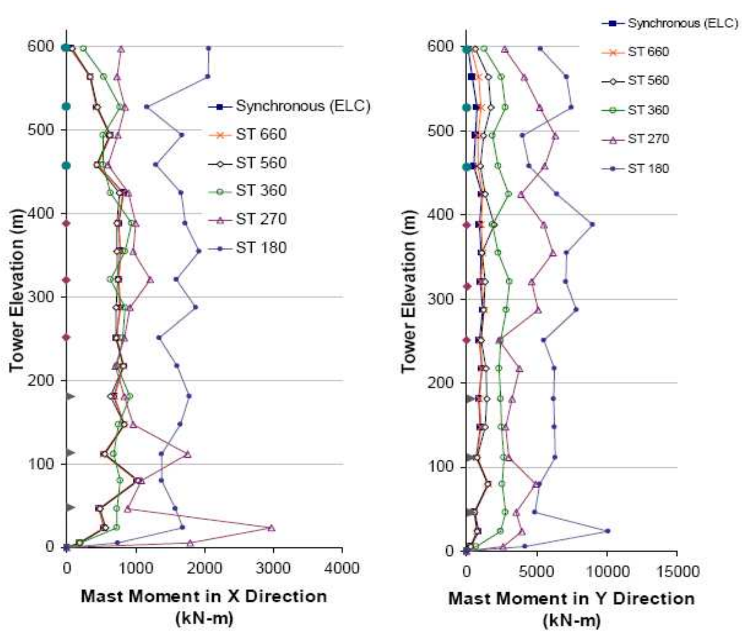

Figure 2-6. Envelope of the bending moment distribution in X and Y directions; El Centro; 607 m mast. [19]

Another interesting finding was that for the three towers studied, the peak response most often occurred in the very beginning of the ground shaking when one side of the tower was vibrating and the excitation had not started in the other parts. Moreover, antenna-supporting structures must meet strict serviceability criteria that depend on their particular function. Seismic amplifications of displacements and rotations will affect the mast during strong shaking, but they should not result in any local permanent deformation if immediate functionality is required after the earthquake. In the study by Faridafshin, all the towers appeared to behave within reasonable serviceability limits in the case of synchronous shaking. But this was not the case under asynchronous excitation.

In the case of the two shorter towers studied, in general, trends in behaviour appear similar to those observed for the 607 m for most of the response indicators. When the shear wave velocity and consequently the time lag between the support excitation increases, greater response is obtained in the structure. However, in the case of the 213 m tower, the effects of asynchronous shaking were not as important or as systematically related to shear wave velocity as in the previous cases.

Throughout the study by Faridafshin, it was confirmed that the displacement-controlled approach for the modeling of earthquake loading has enabled the modeling of asynchronous ground motion in a straightforward manner. The taller the structure, on a softer soil the sensitivity to asynchronous shaking was initiated, and a general trend that could be seen in almost all response indicators in the analyses with asynchronous shaking was the increase in the response when the shear wave velocity decreased and consequently the time lag between the support excitations increased.

3. A review of theoretical developments

3.1. Introduction

The complex interaction between the guy cables and the mast structure is among the most interesting aspects of the dynamic response of guyed telecommunication masts. Although their main contribution is to provide lateral support, guy cables also account for a significant portion of the static and dynamic loads acting on the system. Large compressions as well as horizontal reaction forces are applied to the slender mast by means of the pretensioned guys. Substantial inertial and aerodynamic damping forces are also generated by the motion of the guys which exert a strong influence on the vibration of the entire structure. At the same time, the motion of the mast provides dynamic excitation to the cables.

Due to the critical role of guys in determining the response of the structure, a major emphasis has been placed on an examination of their properties and behavior. In this section, a review of Statics of Suspended Cables is followed by a review of Dynamics of Suspended Cables.

3.2. Statics of Suspended Cables

A catenary profile will be adopted by a cable suspended by its ends under the effect of its weight. This profile will sag below the straight chord line joining its two ends. For a cable with finite extensional rigidity, the distributed load will also generate axial strains, resulting in an elongation of the cable from its original unstrained length.

In guyed towers, as the mast deforms in response to the applied loads, the horizontal component of the tension in the cables H will change in such a way as to oppose the motion. In a suspended cable, two physical mechanisms will contribute to resist relative motion between its end points: (a) the elastic stretching of the cable, (b) the changes to the amount of sag in the cable profile, corresponding to a change in cable slack. The relative contributions of these two mechanisms will vary depending on the degree of the tautness of the cable, for which stretching provides most of the resistance, and the availability of cable slack.

Historical developments in the analytical expressions for the elastic catenary profile are summarized by Irvine (1981) [29]. His theoretical developments will be reviewed in the following sections.

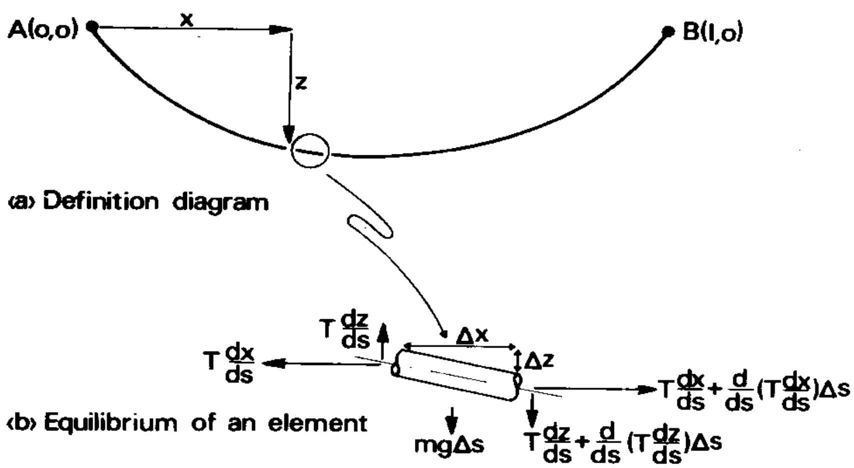

Figure 3-1. Illustration of the horizontal and the vertical equilibrium of an isolated element of the catenary. [29]

The profile adopted by a uniform inextensible cable hanged between two fixed points could be the first approach to the problem. Considering the sketches in Figure 3-1, the vertical and horizontal equilibrium equations of the isolated cable element of length *ds* are:

$$\frac{d}{ds}\left(T\frac{dz}{ds}\right) = -mg \qquad \text{3.1a}$$

$$\frac{d}{ds}\left(T\frac{dx}{ds}\right) = 0 \quad \rightarrow \quad T\frac{dx}{ds} = H \qquad \text{3.1b}$$

The horizontal component of the cable tension is constant everywhere since no longitudinal loads are acting. Consequently, the equation of the vertical equilibrium in Equation 3.1a may be reduced to Equation 3.2.

$$H\frac{d^2z}{dx^2} = -mg\frac{ds}{dx} \qquad \text{3.2}$$

It is seen that, when $mg \; ds/dx$, the intensity of load per unit span (horizontal distance), is constant the resulting profile is parabolic. Enforcing the geometric constraint on arch length, we can take advantage of Equation 3.3.

$$(dx/ds)^2 + (dz/ds)^2 = 1 \qquad 3.3$$

Then, the governing differential equation of the vertical motion will take the form of Equation 3.4.

$$H\frac{d^2z}{dx^2} = -mg\left\{1+\left(\frac{dz}{dx}\right)^2\right\}^{1/2} \qquad 3.4$$

and the solution of this differential equation is the vertical profile *Z(x)*, which also satisfies the boundary conditions. The expression for the length *s* is obtained by integration.

$$Z = \frac{H}{mg}\left\{\cosh\left(\frac{mgl}{2H}\right) - \cosh\frac{mg}{H}\left(\frac{l}{2}-x\right)\right\} \qquad 3.5a$$

$$s = \int_0^x \left\{1+\left(\frac{dz}{dx}\right)^2\right\}^{1/2} dx = \frac{H}{mg}\left\{\sinh\left(\frac{mgl}{2H}\right) - \sinh\frac{mg}{H}\left(\frac{l}{2}-x\right)\right\} \qquad 3.5b$$

If a cable of unstrained length L_0 is used to span between the supports, the horizontal component of cable tension may be found by solving Equation 3.6.

$$\sinh\left(\frac{mgl}{2H}\right) = \frac{mgL_0}{2H} \qquad 3.6$$

In order to calculate the horizontal component of cable tension, H, it is assumed that mg and l are known beforehand. Unless the condition of inextensibility is relaxed, a solution cannot exist if L_0 is not greater than l. The tension at any point along the cable profile is given by Equation 3.7.

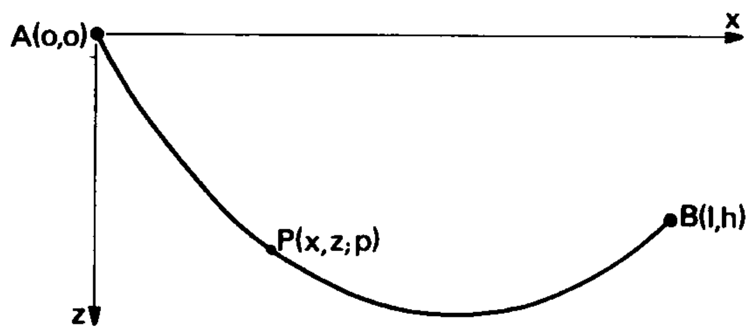

Figure 3-2. Coordinates for the elastic catenary. [29]

$$T = H \cosh \frac{mg}{H}\left(\frac{l}{2} - x\right) \qquad 3.7$$

Furthermore, Irvine [29] adapted inextensible catenary theory to incorporate the elastic effects. The cable shown in Figure 3-2 is suspended between two fixed points A and B, which have Cartesian coordinates $(0,0)$ and (l,h), respectively. The span of the cable is l and the relative vertical displacement of the end points is h. The unstrained length of the cable is L_0, where L_0 is not necessarily greater than $(l^2+h^2)^{1/2}$, although it obviously cannot be much less if Hooke's law is not to be violated. A point on the cable has Lagrangian coordinate S in the unstrained profile. Under the self-weight ($W = mgL_0$), this point moves to occupy its new position in the strained profile described by Cartesian coordinates x and z and Lagrangian coordinate p. Referring to Figure 3-2, the equilibrium of the horizontal and the vertical forces yield to Equations 3.8.

$$T\frac{dx}{dp} = H \qquad 3.8a$$

$$T\frac{dz}{dp} = V - W\frac{s}{L_0} \qquad 3.8b$$

The geometric constraint of Equation 3.3 still applies and the parametric solution describing the strained cable profile can be expressed as follows.

In order to obtain the solution for the tension in the cable $T(s)$, the equations of the vertical and the horizontal equilibriums are squared, added, and substituted into the geometric constraint, to yield Equation 3.9.

$$T(s) = \left\{ H^2 + \left(V - W \frac{s}{L_0} \right)^2 \right\}^{1/2} \qquad 3.9$$

Substituting this expression for T in Equations 3.8 yields the expressions in Equations 3.10 for the x- and the z- components of the cable profile.

$$x(s) = \frac{H\,s}{E\,A_0} + \frac{H\,L_0}{W} \left[\sinh^{-1}\left(\frac{V}{H} \right) - \sinh^{-1}\left\{ \frac{V - W s/L_0}{H} \right\} \right] \qquad 3.10a$$

$$z(s) = \frac{W s}{E\,A_0} \left(\frac{V}{W} - \frac{s}{2L_0} \right) + \frac{H\,L_0}{W} \left[\left\{ 1 + \left(\frac{V}{H} \right)^2 \right\}^{1/2} - \left\{ 1 + \left(\frac{V - W s/L_0}{H} \right)^2 \right\}^{1/2} \right] \qquad 3.10b$$

By satisfying the other end conditions for x and z, the solution for H and T will be implicitly obtained by means of the transcendental Equations 3.11.

$$l = \frac{H\,L_0}{E\,A_0} + \frac{H\,L_0}{W} \left[\sinh^{-1}\left(\frac{V}{H} \right) - \sinh^{-1}\left\{ \frac{V - W}{H} \right\} \right] \qquad 3.11a$$

$$h = \frac{H\,L_0}{E\,A_0} \left(\frac{V}{W} - \frac{1}{2} \right) + \frac{H\,L_0}{W} \left[\left\{ 1 + \left(\frac{V}{H} \right)^2 \right\}^{1/2} - \left\{ 1 + \left(\frac{V - W}{H} \right)^2 \right\}^{1/2} \right] \qquad 3.11b$$

Numerical methods are necessary to solve Equations 3.11 and it has been found that techniques based on the two-dimensional Newton's method are straightforward to implement converge fast.

When the supports are at the same level, $h = 0$, Equation 3.11 (b) yields the expected result of $V = W/2$. Equation 3.12 expresses the horizontal component of the cable tension force in terms of the single dependent variable H:

$$\frac{W}{2H} = \sinh\left(\frac{Wl}{2HL_0} - \frac{W}{2EA_0}\right) \qquad 3.12$$

In order to model a guyed mast, the horizontal stiffness of each guy cable, dH/dx, is generally required. The degree of the lateral restraint provided by a guy cable under static conditions can be expressed in the terms of an effective horizontal stiffness, $k_{xx} = \Delta F_x / \Delta x$.

If the sag is small relative to the cable length, approximately in the cases that the cable sag is less than 1/8 of the cable length, the component of the cable weight, acting perpendicular to the chord line, is approximately constant, and the profile will be parabolic. As such, according to Shears (1968), the horizontal stiffness of a guy in its own plane is given in Equation 3.13.

$$k_x = k_e \left[1 + \frac{w_G \, L^3 \, k_e}{12 \, \bar{T}^3 \left\{ 1 + \frac{8}{3} \left(\frac{\Delta}{L} \right)^2 \right\}} \right]^{-1} \qquad 3.13$$

in which w_G is the weight of the cable per unit length, L is the length of the straight chord line joining its two ends, Δ is the maximum cable sag (perpendicular distance from the chord line), and $\bar{T}$ is the average cable tension which can be calculated through Equation 3.14.

$$\bar{T} = T_B + \frac{1}{2} w_G \, h \qquad 3.14$$

where T_B is related to the bottom end tension and h is the vertical difference between the cable ends. The cable sag Δ is given by Equation 3.15.

$$\Delta = \frac{w_G \, L^2 \cos\theta}{8\bar{T}} \qquad 3.15$$

where θ is the vertical angle between the chord line and a horizontal reference, as shown in Figure 3-3. The variable k_e in Equation 3.13 represents the horizontal stiffness of a perfectly taut wire due exclusively to axial tension, and is defined in Equation 3.16.

$$k_e = \frac{AE}{L}\cos^2\theta \qquad 3.16$$

Since the sag is small for taut cables, the expression of the horizontal stiffness of a taut guy can be simplified to Equation 3.17.

$$k_x \approx k_{EQ} = \frac{1}{\frac{1}{k_e} + \frac{1}{k_g}} \qquad 3.17$$

Where k_g represents the gravitational stiffness that would be provided by an inextensible suspended chain and is defined by the expression in Equation 3.18.

$$k_g = \frac{12\overline{T}^3}{w_G^2 L^3} \qquad 3.18$$

These equations strongly suggest the analogy of two horizontal springs, k_e and k_g acting in series. The parabolic approximation is generally considered to be accurate for cables with sag-to-span ratios, Δ/L_e, of less than $1/8$. This condition is usually satisfied by pre-stressed guy cables used in telecommunication masts.

Using the extensional stress-strain relationship of a parabolic cable, Allsop (1983) proposed a simplified equation to determine the cable stiffness. Further, Dean (1961) had used series expansions of a more exact catenary equation to relate cables end forces and displacements, each of them could be employed instead of Equation 3.17. However, for structural efficiency, guy cables are invariably taut under initial unload conditions. The parabolic approximation, therefore, provides an adequate representation of the practical guy stiffness prior to the application of loads to the system. As the sag-to-span ratio increases, the accuracy of the parabolic approximation deteriorates, and it should be reminded

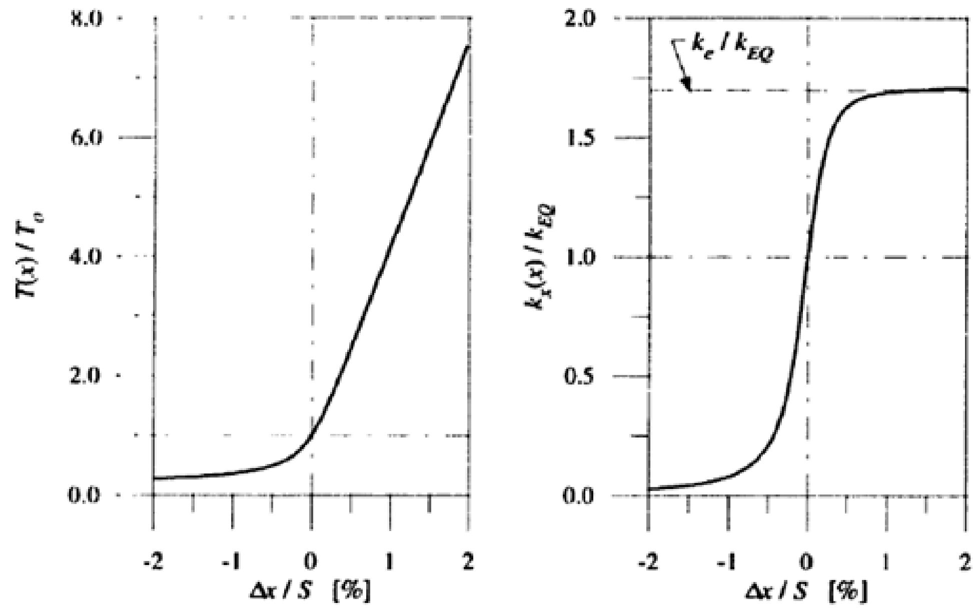

Figure 3-3. Example of static tension and horizontal stiffness curves for single guy [30].

that the best range of efficiency of these equations are when the ration between the sag of the cable and the cable length is less than 1/8.

With the aid of computers, however, efficient iterative techniques can be implemented to solve the catenary cable equation for any arbitrary cable profile. Based on such an iterative solution, Figure 3-3, taken from Sparling (1995) [30], shows the static tension and the horizontal stiffness curves for single guy, with a nominal sag equal to $1/48$ of its chord length; this ratio is well within the accepted range of applicability of the parabolic approximation. Δx is the horizontal displacement of the cable with respect to the initial upright position $(\Delta x = 0)$. As Δx becomes negative, the horizontal cable span of the guy is reduced and its tension declines. For large motions, the tension asymptotically approaches a value equal to the weight of the cable hanging vertically with no pre-tensioning. At this stage, the cable responds to displacements almost exclusively by changes to its profile, with very little stretching taking place. For positive values of Δx, on the other hand, the tension increases rapidly in a nearly linear fashion due to elastic elongation. For positive displacements in excess of $\Delta x / S = 0.5$ %, the sag in the cable has essentially disappeared. The stiffness curve in this region then flattens out and approaches the taut wire stiffness of that cable.

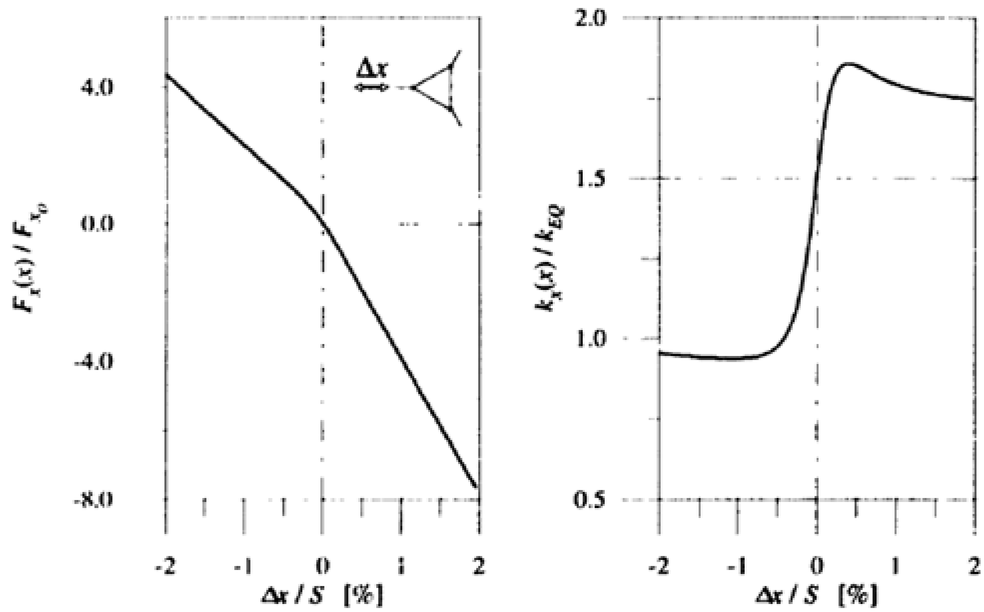

Figure 3-4. Example net horizontal reaction force and stiffness for a group of 3 guys [30].

3.3. Statics of guy clusters

Groups of taut guy wires radiate out from the mast of a guyed tower to provide lateral stability for the system in all directions. The response of the mast is determined by the net reactions from the guy group at each support level. The net horizontal stiffness at a specific guy support level, for small displacements of the mast from its initial position, may be approximated by Equation 3.19.

$$k_{TOT} = \sum_i k_{EQ_i} \cos^2 \beta_i = \frac{N}{2} k_{EQ} \qquad 3.19$$

where k_{EQ_i} is the equivalent horizontal stiffness of the i^{th} guy at that support level, and α_i is the horizontal angle from the direction of interest. For symmetric patterns of guys, we can write the equation in a simplified fashion, in which N is the number of guys at that level. Using the more accurate catenary cable equation, the net horizontal reaction force and stiffness for a group of three cables can be obtained [6].

Guy stiffness values for the unloaded condition, therefore, can be reliably estimated using the expression given above for small deflections. As the mast deflects laterally under horizontal loads, however, the sag rapidly increases in the slacked guys, making the parabolic profile approximation progressively less accurate. At large positive displacements, the net stiffness approaches the elastic stiffness, $k_e \approx 1.7\,k_{EQ}$, for the single guy oriented parallel to the direction of the imposed displacement. At large negative displacements, the net stiffness approaches the combined elastic components of the two guys oriented at 60° to the displacement $2\sum k_e \cos^2 60^\circ \approx 0.85 k_{EQ}$. For large displacements in either direction, the other side guys are largely ineffective in resisting along load motion and the variation of the net horizontal reaction force is bi-linear.

For small positive and negative displacements, both side guys contribute to the net stiffness. As prescribed by small displacement theory, the net stiffness at $\Delta x = 0$ for the three guys considered here is equal to $1.5\,k_{EQ}$.

3.4. Dynamics of Cables and Towers

3.4.1. General

A considerable amount of complexity is introduced into the dynamic modeling of guyed masts by the geometrically non-linear behavior of guy cables. As such, the dynamic behavior of cables deserves more attention.

3.4.2. Natural frequencies and mode shapes

Irvine and Caughey (1974) developed expressions for the natural frequencies and mode shapes of taut, flat cables suspended from two points at the same elevation (level span). Sparling and Davenport (1999) [31] later extended the method to cover inclined cables. For taut cables, which are defined as those having sag-to-span ratios of 1:8 or lower, a distinction is made between modes with shapes that are symmetric about the cable midpoint and those that are anti-symmetric. Based on the parabolic approximation for the cable profile, only symmetric in-plane modes generate axial extension of the cable and, thus, additional cable tension. So, only the symmetric modes are of primary importance with respect to the dynamic response of guyed towers.

In the case of anti-symmetric natural frequencies and mode shapes, the natural frequency of the n^{th} anti-symmetric in-plane mode is defined by Equation 3.20.

$$\omega_n^A = 2n\omega_0; \quad n = 1, 2, 3, ... \qquad 3.20$$

where ω_0 is the fundamental natural frequency of a perfectly taut wire and given by Equation 3.21.

$$\omega_0 = \frac{\pi}{L}\sqrt{\frac{\bar{T}}{m_G}} \qquad 3.21$$

where m_G is the mass per unit length of the guy. The corresponding anti-symmetric mode shape is given by Equation 3.22.

$$\Phi_n^A(x) = \sin\left(\frac{2n\pi x}{L}\right) \qquad 3.22$$

Here x denotes the distance from the lower end of the cable, measured along the chord line and the mode shape is defined as the perpendicular distance from the chord line.

Unlike the anti-symmetric modes, the natural frequencies and modes shapes of the symmetric in-plane modes are influenced by the degree of tautness and axial rigidity of the cable. According to Sparling and Davenport (1999) [31], these effects can be characterized by a single stiffness parameter, λ^2 defined in Equation 3.23 that expresses the relative contributions of the gravitational and the elastic stiffnesses.

$$\lambda^2 = \left(\frac{\omega_G L\cos\theta}{\bar{T}}\right)^2 \frac{\left(\frac{AEL}{\bar{T}}\right)}{L\left[1+8\left(\frac{\Delta}{L}\right)^2\right]} \approx 12\frac{k_e}{k_G} \qquad 3.23$$

Taking the advantage of the compact notation of the stiffness parameter, the natural frequencies and the mode shapes of the symmetric in-plane vibration modes, ω_n^s and $\Phi_n^S(x)$, are defined as the roots of the transcendental Equation 3.24.

$$\tan\left(\frac{\omega_n^S L}{2}\sqrt{\frac{m_G}{\overline{T}}}\right)=\left(\frac{\omega_n^S L}{2}\sqrt{\frac{m_G}{\overline{T}}}\right)-\frac{4}{\lambda^2}\left(\frac{\omega_n^S L}{2}\sqrt{\frac{m_G}{\overline{T}}}\right)^3 \qquad 3.24a$$

$$\Phi_n^s(x)=1-\tan\left(\frac{\omega_n^S L}{2}\sqrt{\frac{m_G}{\overline{T}}}\right)\sin\left(\omega_n^S\sqrt{\frac{m_G}{\overline{T}}}\,x\right)-\cos\left(\omega_n^S\sqrt{\frac{m_G}{\overline{T}}}\,x\right); \qquad 3.24b$$

$n = 1, 2, 3, ...$

For taut cables, which are associated with low values of λ^2, the smallest natural frequency is the first symmetric mode (ω_1^s); consequently, the symmetric and the anti-symmetric modes alternate. As the sag in the cable increases, which is associated with larger values of λ^2, the symmetric natural frequencies increase; at high λ^2, they are approximately $2\omega_0$ above their value at low λ^2, while the anti-symmetric frequencies remain essentially constant.

Figure 3-5 depicts the variations of the in-plane natural frequencies of a guy versus λ^2. At some point, the corresponding symmetric and anti-symmetric

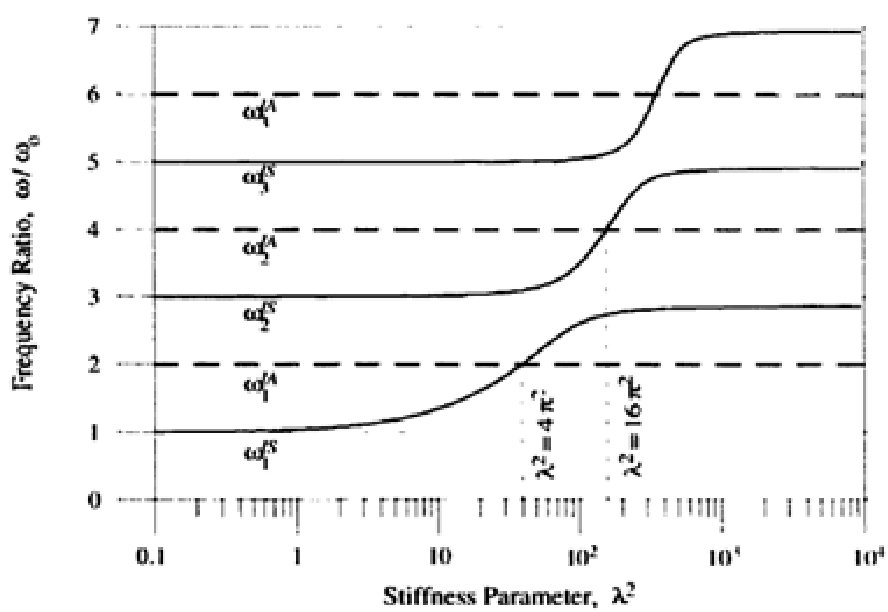

Figure 3-5. Variation in guy in-plane natural frequencies with λ^2. [6].

frequencies will be equal. This state is referred to as the cross-over point, and it occurs for the n^{th} pair of natural frequencies when the stiffness parameter is equal to $\lambda^2 = (n+1)^2 \pi^2$. Since modal coupling at these cross-over points allows the transfer of energy from the anti-symmetric modes, which do not generate additional tension, to the symmetric modes, which do, the potential for increased dynamic excitation of the mast exists.

For the symmetric modes, the variation of the natural frequency is accompanied by the corresponding changes in the mode shapes. The lowest symmetric modes changes from that of a taut wire, which is approximately sinusoidal at low λ^2 values, to one resembling that of an inextensible suspended chain, which eventually developing two internal nodes for large sags. The shapes of the first symmetric in-plane mode for three values of λ^2 are illustrated in Figure 3-6.

The implication of the symmetric mode shape variations for the extensional resistance of a cable were first discussed by Veletsos and Darbre (1983). Ahmadi-Kashani (1989) also found that various existing analytical expressions could be used to describe cable variations over specific ranges of sag-to-span ratios. Therefore, it seems quite rational to develop analytical expressions for cable dynamic characteristics which are appropriate to specific structural applications, including guyed telecommunication towers.

In the case of out-of-plane vibrations, there is no distinction between the symmetric and the anti-symmetric modes. Therefore, the out-of-plane undamped natural frequencies for the n^{th} out of plane vibration mode maybe expressed as:

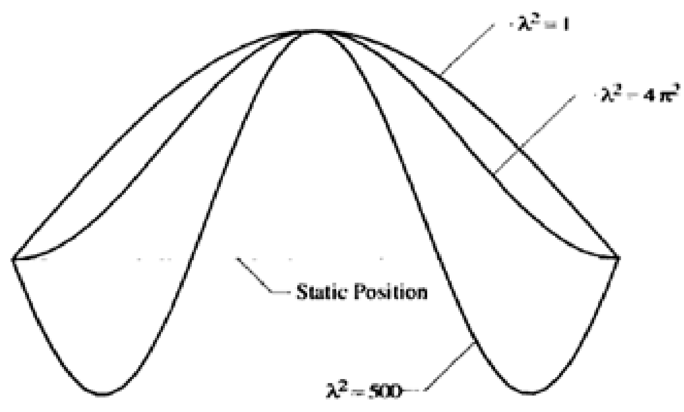

Figure 3-6. Variation in the first symmetric mode shape with λ^2. [6].

$$\omega_n^0 = n\,\omega_0\,; \qquad n = 1,\,2,\,3,\,... \tag{3.25}$$

where ω_n^0 is the fundamental taut wire frequency. The associated out-of-plane mode shapes, $\Phi_n^0(x)$, are

$$\Phi_n^0(x) = \sin\left(\frac{n\,\pi\,x}{L}\right) \tag{3.26}$$

3.4.3. Dynamic cable stiffness

The inertial effects associated with the vibration of the cable mass have a dominating effect on the restraint offered by a guy cable to resist the horizontal motion of the mast as well as on the damping, and the dynamic cable stiffness is highly dependent on the frequency of the imposed motion.

For frequency domain analyses, the displacement of the mast and the corresponding horizontal resisting force supplied by the guy cable can be related using a complex-valued cable stiffness parameter, consisting of two components, namely a real value component $K^{\mathrm{Re}}(\omega)$, which describes the in-phase component of the cable resisting force, and an imaginary component $K^{\mathrm{Im}}(\omega)$, which describes the out-of-phase force component that does not contribute to resistance but instead relates to the energy dissipated from the system. According to Davenport and Steels (1965), based on the linear behavior and the parabolic cable profile, if the upper end of the cable is subjected to horizontal displacements at a forcing frequency ω, the dynamic cable stiffness components are given by Equation 3.27.

$$K^{\mathrm{Re}}(\omega) = k_e\left[1 - \frac{\left(F\,\Omega^2 - 1\right)\left(G\;\gamma_1(\Omega) - 1\right) + F\;G\;\Omega\;\xi\;\gamma_2(\Omega)}{\left(G\;\gamma_1(\Omega) - 1\right)^2 + \left(G\;\gamma_2(\Omega)\right)^2}\right] \tag{3.27a}$$

$$K^{\mathrm{Im}}(\omega) = k_e\left[\frac{F\,\Omega\,\xi\left(G\;\gamma_1(\Omega) - 1\right) + \left(F\;\Omega^2 - 1\right)G\;\gamma_2(\Omega)}{\left(G\;\gamma_1(\Omega) - 1\right)^2 + \left(G\;\gamma_2(\Omega)\right)^2}\right] \tag{3.27b}$$

where Ω is the non-dimensional frequency ratio defined by Equation 3.28.

$$\Omega = \omega / \omega_0 \ , \quad where\ \omega_0 = \frac{\pi}{L}\sqrt{\frac{\overline{T}}{m_G}} \qquad 3.28$$

and ω_0 is the fundamental natural frequency of a taut wire, $k_e = AE/L \ \cos^2\theta$ is the elastic cable stiffness, and ξ is the viscous damping ratio defined as a fraction of the critical damping. The additional parameters are defined in 3.29.

$$F = \frac{\pi^2 \sin\theta\, \overline{T}^2}{2\, w_G\, L^2\, k_e} \qquad G = \pi^2 \frac{\overline{T}^3}{L^3\ w_G^2\ k_e} \qquad 3.29a, b$$

$$\gamma_1(\omega) = \frac{\pi^2}{8}\left(\frac{\psi_1}{\psi_1^2 + \psi_2^2}\right) \qquad \gamma_2(\omega) = \frac{\pi^2}{8}\left(\frac{\psi_2}{\psi_1^2 + \psi_2^2}\right) \qquad 3.29c, d$$

$$\psi_1 = \sum_{n\,odd} \frac{\Omega^2 - n^2}{n^2[(\Omega^2 - n^2)^2 + (2\zeta\Omega)^2]} \qquad \psi_2 = \sum_{n\,odd} \frac{2\zeta\Omega}{n^2[(\Omega^2 - n^2)^2 + (2\zeta\Omega)^2]} \qquad 3.29e, f$$

in which n is the number of terms included in the Fourier series used to approximate the dynamic cable displacement about its static profile.

Based on the described model, the typical frequency-dependent characteristics of the dynamic stiffness modulus of a taut cable are shown in Figure 3-7. The real component of the stiffness $k^{\mathrm{Re}}(\omega)$ initially decreases with increasing frequency, starting from its static stiffness value of k_{stat} at low frequencies. A negative stiffness signifies that the cable is pulling the mast in the direction of motion rather than offering resistance. At the first symmetric natural frequency, the sign of $k^{\mathrm{Re}}(\omega)$ reverses suddenly, becoming strongly positive, and would in theory tend to infinity in the absence of damping. This resonant behavior is also evident over a narrow frequency range at the cable second symmetric natural frequency $(\omega \approx 3\omega_0)$. At the higher modes, $k^{\mathrm{Re}}(\omega)$ gradually approaches the cable elastic stiffness value of k_e, suggesting that the cable is responding to high frequency motion by stretching only and acting much like a straight, massless rod. At each of the higher symmetric natural frequencies of the guy, the $k^{\mathrm{Re}}(\omega_0)$ curve again drops suddenly and then jumps to a large positive value before again returning quickly to k_e. However, the width of the resonant frequency band becomes increasingly narrow as the mode number increases.

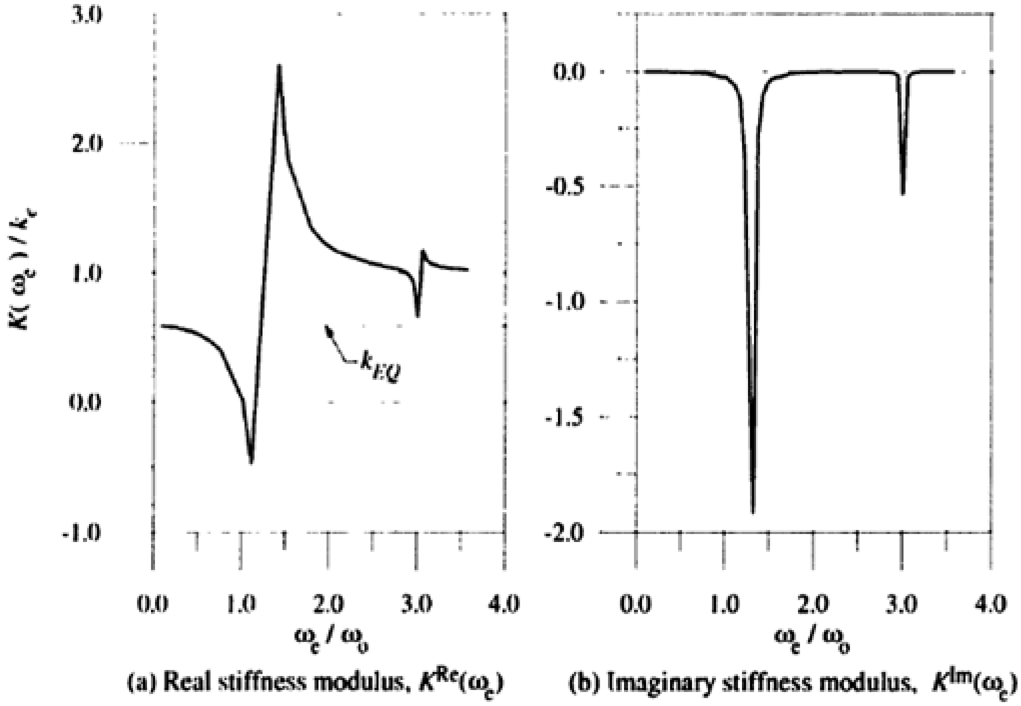

Figure 3-7. Complex guy stiffness vs. frequency [6].

Experimental and numerical studies indicated that only the lowest mode is of significance importance since higher symmetric modes are difficult to activate. In addition, out-of-plane swaying motions were observed at the first and third resonant frequencies, occurring at a frequency of one-half that of the in-plane motion.

The imaginary stiffness component $k^{\mathrm{Im}}(\omega)$ is only significant at resonant frequencies, where it takes on large negative values, which indicates that energy is being dissipated. These spikes become increasingly narrow for the higher modes. They indicate that there is a sudden change in the phase shift between the motion of the mast and the resisting force of the guy cable at these frequencies. For this linear model, resonance of the antisymmetric modes has no effect on either component of the cable stiffness.

Although informative and suitable for some types of dynamic analysis, the frequency-dependent description presented here cannot be used with time-

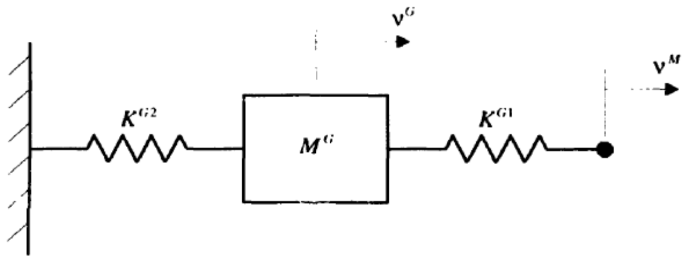

Figure 3-8. Equivalent spring-mass guy model.

domain modal analysis methods, since these require frequency-independent properties.

To overcome this difficulty, Davenport and Hartmann (1966) proposed a simplified spring-mass-spring cable model, shown in Figure 3-8. The model was later improved by adding damper elements, structural and aerodynamic. For a cable vibrating in its own vertical plane in the fundamental symmetric mode, the equivalent properties are presented in Equations 3.30.

$$K^{G1} = k_e \left[1 - \frac{4\, w_G\, L \sin\theta}{\pi^2\, \overline{T}} \right] \qquad 3.30a$$

$$K^{G2} = \frac{K^{G1}}{\dfrac{4\, w_G\, L}{\pi^2\, \overline{T}} \left[\dfrac{2\, g\, k_e}{\omega_0^2\, \overline{T}} - \sin\theta \left\{ 1 + \dfrac{8}{\pi^2} \left(\dfrac{w_G\, L}{\overline{T}} \right) \left(\dfrac{g\, k_e}{\omega_0^2\, \overline{T}} \right) \right\} \right]} \qquad 3.30b$$

$$M^{G} = \frac{K^{G2}}{\omega_0^2} \left(1 - \frac{4\, w_G\, L \sin\theta}{\pi^2\, \overline{T}} \right) \qquad 3.30c$$

As it seems that the frequency of the vibration of the cable clusters has the most dominating effect on the dynamic stiffness of the cable clusters, Equations 3.30 worth more attention when it comes to estimate the dynamic stiffness of cable clusters.

4. A review of Code developments

4.1. General

The dependency of communication networks on very tall broadcast guyed towers has made them strategic components of the system, especially in the case of post-disaster response. Therefore, their preservation in the event of a severe earthquake could be critical. Although it is probable that they would not be serviceable during major strong motion, they must not experience important damage in order to resume its function shortly after the event. As such, owners of telecommunication towers are motivated to assess the seismic performance of their towers. At the same time, structural designers of telecommunication towers receive very little guidance from their national standards for seismic analysis, and there is still a need for further investigation in this field.

Consequently, in the last decade, there has been an increased interest in North America, as well as Europe, in establishing earthquake resistance design guidelines specifically for communication structures. The primarily purpose of these design codes and guidelines is that tower designers would be able to satisfy the local building control requirements, where general seismic design criteria for buildings would not be appropriate for communication structures.

In this section, a review of the general recommendations for the seismic analysis of guyed masts by IASS-WG4 [32] will be presented first. Considering the recent developments of the North American Telecommunication Codes, the Canadian Code [33] and the American Code [34] for telecommunication towers will be reviewed in more details.

4.2. Recommendations of IASS-WG4 for the seismic analysis of guyed telecommunication masts

The International Association for Shell and Spatial Structures, IASS-WG4, published very general recommendations for the analysis and design of guyed masts in 1981 [32]. Only the main guidelines relevant to seismic analysis are summarized here.

This report suggested that a simplified seismic analysis of guyed towers be done using a static lateral load proportional to their weight (like most building codes). It recommended several simplifying assumptions in order to linearize the analysis and to use modal superposition. Simple linear springs were suggested to represent the guys, which were associated with lumped masses representing the inertia effects of the cables. However, it mentioned caution in the use of this spring-mass guy cable model for tall masts, due to significant geometric nonlinearities introduced in slackening guys.

Since seismic design loads represent extreme events, their combination with the dead loads only was suggested, with the assumption that earthquakes occur in still air conditions. IASS-WG4 further recommended (with no detailed guidance) using random vibration approaches in seismic load modeling and neglecting the ground surface wave propagation effects.

The recommendations by IASS-WG4 in 1981 were really a ground breaking approach to the dynamic analysis of guyed telecommunication towers. However, the important advances in theoretical and computational structural dynamics that have occurred since then have called for a vast revision of the report, which has been replaced by Smith (2007) [1].

4.3. Canadian Standard CSA-S37-01(R2006), Antennas, Towers, and Antenna-Supporting Structures

4.3.1. Introduction

Guidance on earthquake-resistant design and seismic analysis of communication structures is contained in Appendix M of the Canadian standard CSA-S37-01(R2006) [33]. It is not a mandatory part of the Standard. These guidelines were first introduced in the 1994 edition (Appendix L) and then revised and augmented in 2001: the 2001 version of the standard has been reaffirmed with no change in 2006. Note that Appendix M should be revised soon to reflect the changes in Canadian seismic hazard introduced in the 2005 edition of the National Building Code [16].

According to CAN/CSA S-37 [33], earthquake effects should be considered for susceptible towers of critical importance, e.g. post-disaster communication systems, in high-risk earthquake zones.

4.3.2. Seismicity and earthquake-resistance performance levels

There are three main considerations which dictate the seismic design requirements of a telecommunication structure, namely: the seismicity of the structure's location, the geotechnical aspects of the site, and the performance level required by the owner for the structure. The definition of earthquake-resistance performance levels was first introduced in the 2001 edition of the Code to address this last consideration.

The selection of an appropriate reliability level is a key design decision in structural engineering. In building design, the main concern of seismic provisions is to provide an acceptable level of safety against injury or loss of life. Codes will generally provide the design earthquake parameters corresponding to an acceptably low probability of exceedance, for instance not more than 10% in 50 years. Although the risk to injury or loss of life is also a first and foremost concern in tower design, there are other concerns depending on the economical value and the function of the structure. The tower owner (telecommunication service provider) should decide on the appropriate reliability level, generally quantified in terms of appropriate partial loading factors. In general, properly designed masts should resist moderate earthquakes without significant damage and major earthquakes without collapse. However, if the structure is part of a post-disaster communication network, its damage or loss of functionality may amplify the number of fatal casualties and compromise the rescue effort. The Canadian Code uses three categories to define the safety level of the structure: prevention of injury or loss of life, interrupted serviceability, and continuous serviceability.

In the Canadian Code, all structures located in high seismicity areas, $\left(PGA \geq 30\%\, g\right)$- *Peak Ground Acceleration*-, should be checked to insure that no injury or loss of life may result from the collapse of the structure. Structures located in low seismicity areas, $\left(PGA < 15\%\, g\right)$, need no seismic design precaution. In moderate seismicity areas, the effect of earthquake loads should be considered in design of all structures supported on buildings and of structures where continuous serviceability has to be maintained. Furthermore, a seismic design check on the structures of interrupted serviceability is recommended for unusually configured structures or in guyed masts taller than 300 m.

4.3.3. Prediction of seismic response of structures

The coincidence between the dominant natural frequencies of the structure and the frequency content of the excitation will extensively influence the seismic sensitivity of the structure. Investigations indicate that past earthquake records have typical dominant frequencies in the range of *0.1* to *10 Hz*, with a concentration in the range of *0.3* to *3 Hz* for the horizontal motions, while the vertical motions involve a higher frequency band. Therefore, the first step in the assessment of the tower sensitivity to earthquakes is the evaluation/prediction of the dominant natural frequencies of the structure.

It is worthy of note that CSA S37 recommends the dynamic analysis of self-supporting lattice structures only for the 50-m and taller towers requiring continuous serviceability is active seismic zones. In addition, special caution is recommended for irregular towers, towers supported by building, and towers mounted by excessively heavy head loads.

However, in the case of guyed telecommunication towers, most masts with height ranging typically from 150 m to 300 m have their fundamental flexural frequencies within the sensitive range of the frequency content of the excitation. Nonetheless, seismic effects are not likely to govern the design (wind effects will be more critical) in areas with moderate seismic hazard. The Code mentions that mast-guy dynamic interaction could be, potentially, an important phenomenon of response amplification. This may be worthy of investigation when the vertical ground motion is combined with the usual horizontal one, provided that there is a frequency coincidence between the input dominant frequencies and the frequencies of the dominant strongly-coupled cable and mast modes. Furthermore, the study of the earthquake effects requires modeling of the seismic input in terms of the prescribed components of the ground displacements along the three orthogonal directions at each support and with an appropriate correlation. Earthquake effects on the masts itself appear to be significant only in the top cantilevered portion (if present) of tall masts and in the first span near the base. Also, dynamic amplifications in the guy tensions are more likely to be significant in the top and the bottom levels of the multilevel guyed masts. Those relatively slack cables with initial tension below about 5% of their UTS, although rarely used in practice, are potentially more vulnerable than taut cables.

Rational simplified and quasi-static analysis models for guyed masts under seismic actions have not been proposed in the literature, unlike for wind actions. Therefore, CSA-S37 recommends to perform a detailed dynamic analysis for all masts of height above 150 m located in high seismicity areas and for all masts where continuous serviceability is needed in moderately active areas. Structures taller than 300 m should be subjected to a detailed analysis where there is a risk of injury or loss of life in moderately active areas, including the effect of asynchronous motion at the mast base and stay anchorages. But all of the studies forming the research basis of Appendix M of the Canadian Standard have shown that detailed nonlinear seismic analyses are far more complex than response spectrum analysis, and not always necessary. Calculation of the natural frequencies of the initial tower configuration can help to identify the seismic sensitivity of the structure and potential interaction effects due to clustered frequencies. However, for towers with a confirmed potential for seismic sensitivity, it is clear that a rational simplified procedure for seismic design would be very useful.

4.3.4. Other considerations

Appendix M of CSA-S37 recommends obtaining a site-specific geotechnical report for communication structures of height of 50 m and above planned in active seismic areas, as well as for masts in areas with moderate hazard when there is a risk of injury or death if the structure collapses. Special consideration should be given to structures located near an active fault, as well as those that face a risk of soil settlement, consolidation, liquefaction, flooding, and foundation sliding or pile failure due to ground motion.

As for the influence of antennas and ancillary components attached to the tower structure, parametric analyses of self-supporting lattice towers have shown that it is not significant unless the additional concentrated masses are in the order of more than 5% of the total mass of the primary structure. This observation cannot be directly extrapolated to tall masts, where the particular location of additional lumped masses with respect to the guy attachment levels also becomes a factor. The effect of the distributed mass of transmission lines, ladders and other attachments can easily be accounted for in models with the increased effective mass of the mast structure.

4.4. American Standard ANSI/TIA/EIA-222-G on Seismic design provisions

4.4.1. Introduction

Following the Canadian initiative and along with worldwide seismic awareness, the American Electrical and Telecommunication Industries Association – EIA/TIA (TIA/222G Telecommunication Industry Standard, 2005) [34] has also established earthquake resistance design guidelines specifically for communication structures, primarily to satisfy local building control requirements. The objective of this standard is to provide recognized literature and minimum design requirements for antenna-supporting structures and antennas for all classes of communications service, such as AM, FM, microwave, wireless, TV, etc.

According to TIA/222G, earthquake effects may be ignored for structures presenting a low hazard to human life or damage to property in the event of failure. Moreover, structures used for services that are optional or where a delay in returning the service would be acceptable, such as residential wireless, television, radio, wireless cable, amateur and CB radio communication, maybe designed without any seismic considerations. In addition, seismic design may be ignored for any structure located in a region of low seismicity level. Furthermore, for towers without torsional, stiffness, and mass irregularities earthquake effects may be ignored when the total seismic shear force is less than 75% of the total horizontal wind load without ice. However, in the all other cases, earthquake effects must be considered in the design of telecommunication towers. It should be noted that the seismic requirements of TIA/222G are mandatory, while the Canadian guidelines are not.

4.4.2. Selecting an appropriate importance factor

The first step in the seismic analysis of telecommunication towers is to assess the level of the importance of the structure. According to TIA/222G, there are several considerations such as height, use, or location of the tower, as well as whether it represents a substantial hazard to human life and damage to property in the event of failure. Structures used for essential communications, such as civil and national defense, emergency, rescue or post-disaster operations, military and navigation facilities, receive the highest level of importance.

4.4.3. Selecting an appropriate seismic analysis method

TIA/222G lists four seismic analysis methods:

Method 1	Equivalent static lateral force
Method 2	Equivalent modal analysis
Method 3	Modal analysis
Method 4	Time history analysis

However, Methods 2 and 3 are not strictly applicable to guyed masts who exhibit nonlinear response under strong ground shaking. The equivalent lateral force, Method 1, is deemed applicable to guyed masts with maximum height of 450 m (maximum ground anchor distance of 300 m for guy cables) and without mass or stiffness irregularities. It should be noted that vertical seismic forces may be ignored in the first three methods.

Considering the limitations and the deficiencies of Method 1, time history analysis, Method 4, is the alternative for all towers and masts with or without irregularities. In this detailed approach, vertical components of seismic excitation as well as spatial variations can be explicitly considered in the analysis.

Equivalent lateral force procedure (TIA/222G Method 1)

In the equivalent lateral force procedure, the total horizontal seismic shear force is calculated and further distributed along the height of the mast. Consequently, the structure will be analyzed statically using these equivalent seismic forces as external loads. It is worthy of note that in order to determine the total weight of the structure, all appurtenances and one-half of the guys' weight should be included. The total seismic shear, V_S, is calculated according to Equation 4.1.

$$V_S = \frac{S_{DS} WI}{R} \qquad 4.1$$

Alternately, for ground supported structures the total seismic shear don't need to be greater than, $V_S = \frac{f_1 S_{D1} WI}{R}$, but not less than $0.044\, S_{DS}\, WI$, and for sites where S_1 is equal or exceeds 0.75, V_S shall also not be less than the value in Equation 4.2.

$$V_S = \frac{0.5\, S_1 WI}{R} \qquad 4.2$$

where:

S_{DS} : Design spectral response acceleration at short periods

S_{D1} : Design spectral response acceleration at a period of 1.0 second

S_1 : Maximum considered earthquake spectral response acceleration at 1.0 second

W : Total weight of the structure including appurtenances, for the guyed masts also includes one-half the weight of guys supporting the structure

I : Importance factor

R : Response modification coefficient equal to 2.5 for latticed guyed masts.

and finally f_1 : the fundamental frequency of the structure, involving the vibration of the mast. In lieu of a rational analysis, the fundamental natural frequency of a guyed mast may be determined through Equation 4.3.

$$f_1 = C_g \sqrt{\frac{K_g}{W_t}}$$

$$K_g = \sum_{i=1}^{n} \left[\frac{N_i \left(A_{gi}\right)\left(G_{ri}\right)\left(H_{gi}\right)}{h \left(L_{gi}\right)^2} \right] \qquad 4.3$$

where C_g is equal to *176.5* for English units, K_g is the equivalent horizontal stiffness of the guying system, W_t is the weight of the structure including appurtenances and the total weight of all guys (remember that W is the total weight of the structure including appurtenances and one-half the weight of guys supporting the structure), n is the number of guying levels, N_i, A_{gi}, G_{ri}, H_{gi} are the number of guys, the cross-sectional area of individual guys, the average guy radius, and the height above base, respectively, for the i^{th} guy elevation. Finally, H is the height above the base to the top of the mast (excluding appurtenances). Alternately, the simpler empirical Equation 4.4 may be used:

$$f_1 = K_m \sqrt{\frac{1}{h^{1.5}}} \qquad 4.4$$

where K_m is equal to 122 for h , the tower height excluding appurtenances, in feet.

In order to distribute the seismic forces along the height of the mast, the lateral seismic force F_{SZ} at any level z shall be determined from Equation 4.5.

$$F_{SZ} = \frac{W_Z h_Z^{ke}}{\sum_{i=1}^{n} W_i h_i^{ke}} V_S \qquad 4.5$$

where W_z , W_i and h_z , h_i are the portions of total weight and the height from the base of the structure to level *z,* respectively. k_e is the seismic force distribution exponent, which is equal to 1.0 (linear) for structures having a fundamental frequency of 2.0 Hz or higher and equal to 2.0 (parabolic) for structures having a fundamental frequency of 0.4 Hz or less. In the author opinion, the method introduced here is identical to the theory of cantilever shear beam models recommended by building seismic codes, and the direct applicability of this approach to guyed masts needs more attention. Guyed masts do not behave as free-standing cantilevers, and these parts of telecommunication structures codes should be validated throughout the deep study on the physics of the behavior of the structures.

Time history analysis (TIA/222G Method 4)

The time history analysis (Method 4) of TIA/222G recommends that the mathematical model of the guyed mast represent the spatial distribution of the mass and the stiffness throughout the structure, including structural damping equivalent to 5% of critical viscous damping. The mathematical model should be able to consider the inertia and the damping of the cables properly.

The procedure for scaling the input ground motion is presented in details. Firstly, one vertical and two orthogonal horizontal ground motion time histories, from not less than three different recorded events representative of the seismicity of tower site, should be selected. Then, for each horizontal component, a 5%-damped response spectrum is constructed, and the response spectra for each pair of horizontal components are to be combined, using the square root of sum of squares (SRSS). Consequently, the average of the resulting combined spectra should be calculated. Finally, the horizontal ground motion components are scaled such that the averaged combined spectrum is not less than 1.3 times the design response spectrum, calculated in accordance with the code multiplied by the importance factor for the structure. It is also worthy of note that the scaling factor shall be applied to all three ground-motion components. Consequently, a time-history analysis for each event [of input time histories] in accordance with acceptable methods of structural analysis is performed.

There are a couple of minimum acceptable modeling considerations for guyed masts in TIA/222G. For instance, the mast model can be either an elastic three-dimensional beam-column mast, an elastic three-dimensional truss model or a three-dimensional frame-truss model. In addition, the analysis shall take into account the global $P-\Delta$ effects on the mast induced by its sway motion. Also, for guyed telecommunication masts, the effects of out-of-phase excitation of the anchor point shall be included in the analysis.

Serviceability requirements

According to TIA/222G, the mast translations (sway) and rotations (twist and tilt) under service loads shall comply with some serviceability limits, unless otherwise required.

If precise serviceability criteria are not specified by the tower owner/service provider, TIA/222G lists a few general minimum requirements that apply to the mast: The tilt (flexural rotation) should not exceed 4° about the vertical axis; the twist (torsional rotation) should not exceed 4° about any horizontal axis of symmetry, and the maximum sway displacement is limited to 5% of the tower height (excluding appurtenances installed at the tower top).

The operational twist and sway limits of the mast at the elevation of an antenna shall be calculated in accordance with the antenna specifications. According to TIA/222G, for a parabolic reflector with allowable radio frequency signal degradation of *10dB* and *3dB,* the twist limits, denoted as θ are taken as $\theta = C_{10}/D\alpha$ and $\theta = C_3/D\alpha$, respectively, with $C_{10} = 16.2$ and $C_3 = 9.45$. The other parameters are D, the antenna (drum or dish) diameter, and α, the antenna operating frequency in GHz.

5. Review of other works

There are not many published reports in the open scientific literature on the seismic behavior of telecommunication towers, especially if compared to the abundant seismic literature on building structures. However, even if the number of researchers in the field is limited, several appreciable developments have been made in the more general area of dynamics, especially under wind loading. In this section, this research will be reviewed. Special attention will be first paid to Canadian studies, followed by the contributions made in the United States, and finally the review will be completed by other international studies.

5.1. Recent research on guyed communication towers in Canada

5.1.1. Introduction

Besides the research reviewed in Chapter 3, associated with work done at McGill University, a few other studies have been completed in Canadian research centers in dynamic analysis of guyed telecommunication towers.

Work at the University of Western Ontario resulted in the derivation of a simplified dynamic analysis procedure for masts subjected to turbulent wind effects; it was reported in Sparling (1995) [30] and Sparling and Davenport (1999) [31]. This research will be reviewed in some details.

Work at the University of Windsor includes a study by Wahba (1999) [15] on the general dynamic properties of guyed masts, and a more recent one by Meshmesha (2005) [35] on seismic design of tall masts, which will be reviewed last.

5.1.2. Simplified analysis methods for telecommunication masts in turbulent winds

Since telecommunication towers are typically quite light, wind loads often control the design for lateral effects. Therefore, a reliable tower design is highly dependent on an accurate assessment of the response of the structure to wind. However, the detailed dynamic analysis of a guyed mast subjected to strong turbulent wind is complex, mainly due to the nonlinear response of the system and the random nature of the wind loads. There are significant differences between the static and the dynamic response characteristics of guyed telecommunication towers, and conventional static analysis methods can lead to unsafe designs for wind effects.

There is always the option of performing a detailed nonlinear dynamic study of the structure using sophisticated finite element methods. However, outside research environments, it is rarely done because of the complexity of these approaches. In engineering practice, simplified quasi-static methods are always appreciated because there is limited time and budget devoted to any specific tower design. A well-known simplified dynamic analysis method – the patch load method - for guyed masts in turbulent winds has been developed by Sparling (1995) [30], and is now accepted by a few design codes, including the Canadian CSA S37 code and the British standard, BS 8100 Part 4 for guyed masts [36]. The method is described next.

Considering a typical response time history of a guyed mast to wind loads, as shown schematically in Figure 5-1 (a), Sparling assumed that the total response of the structure r fluctuates randomly about a mean (time-averaged) value $\bar{r}$, as stated in Equation 5.1.

$$\tilde{r}(t) = r(t) - \bar{r} \qquad where \quad \bar{r} = \frac{1}{T}\int r(t)dt \qquad 5.1$$

where T is the period over which the response sample is defined and $\tilde{r}(t)$ is the fluctuating component of the response. By definition, $\tilde{r}(t)$ has a mean value of zero and its mean square value $\tilde{r}^{2}$ is given by Equation 5.2.

$$\tilde{r}^2 = \frac{1}{T}\int \{r(t) - \bar{r}\}^2 dt \qquad 5.2$$

The mean response component is determined by nonlinear static analysis of the mast under the mean component of the wind load. This wind load is defined in Equation 5.3.

$$\bar{F}(z) = \frac{1}{2}\,\rho_a\; C_D(z)\; A(z)\,\bar{v}(z)^2$$
$$F = \frac{1}{2}\,\rho\; C_D\;\; A\,U^2 \qquad 5.3$$

In which z is the elevation of the point in question, ρ_a is the density of the air (approximately 1.25 kg/m3), $C_D(z)$ is the effective coefficient of the drag of the structure and ancillaries, $A(z)$ is the effective projected area of the structure and ancillaries, and $\bar{v}(z)$ is the mean wind speed at the elevation of the point in

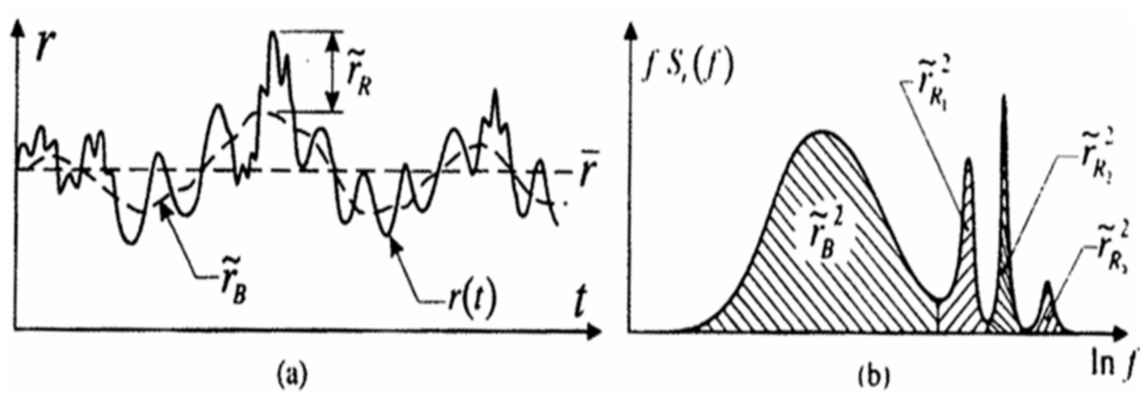

Figure 5-1. The response of a guyed mast to wind; (a) time history, (b) Power spectra. [37]

question. The distribution of $\tilde{r}^2$ in terms of the frequency content is shown schematically in the power spectral density function form on Figure 5-1 (b). The fluctuating component of the response may be further subdivided into a slowly varying background response $\tilde{r}_B$ and a more rapidly varying resonant response $\tilde{r}_R$. The power spectrum indicates how the energy of the fluctuating response is distributed with frequency; the area under the power spectrum is the magnitude of the mean-square fluctuating response. However, the background response occurs over a broad band of frequencies in the low frequency range, below the fundamental frequency of the mast. In this model, the dynamic amplification effects in the background response are therefore negligible, and it is sufficient to retain only the quasi-static response of the system.

In addition, since all low frequency response is included in the background component, there is no need to consider the coupling between the tower vibration modes over this frequency range. Finally, the sensitivity of the analysis to inaccuracies in the shape of the low frequency range of the wind spectrum is reduced. The determination of the background response is based on the linear static stiffness properties of the system calculated at the mean equilibrium position. The rms value of the fluctuating wind load $\tilde{F}(z)$ used in background response calculations may be expressed as:

$$\tilde{F}(z) = \rho_a \, C_D(z) A(z) \bar{v}(z) \, \tilde{v} \qquad 5.4$$
$$where \quad \tilde{v} = i_0 \, \bar{v}_{ref}$$

in which $\tilde{v}$ is the rms fluctuating wind speed and i_0 is the turbulence intensity factor that depends on the site condition $0.18 \sim 0.27$.

The resonant response is composed of a series of narrow peaks centered on the natural frequencies of the structure. It is calculated using modal superposition techniques. Based on the system properties at the mean equilibrium position, an eigenvalue analysis is performed to determine the natural frequencies and generalized modal properties of the mast (lowest 15 to 30 modes). Consequently, generalized modal forces are based on the wind spectrum and account for uncorrelated fluctuating loads by employing a narrow band correlation function.

If the damping forces are small and the natural frequencies of the system are well separated, the cross-coupling between the vibration modes will be negligible. In that case, there will be a little overlap between the individual resonant peaks in the spectrum, and the resultant resonant response may be determined by adding the squared values together and taking the square root of the resulting sum.

$$\tilde{r}_R^{\,2} = \sum_j \tilde{r}_{R_j}^{\,2} \qquad 5.5$$

The resultant magnitude of the total root-mean-square (rms value) fluctuating response $\tilde{r}$ is given by Equation 5.6.

$$\tilde{r} = \sqrt{\tilde{r}_B^{\,2} + \tilde{r}_R^{\,2}} \qquad 5.6$$

in which $\tilde{r}_{R\,j}$ is the rms resonant response in the $j\,th$ mode of vibration. For design purposes we can take advantage of Equation 5.7.

$$\hat{r} = \bar{r} + g\,\lambda_B\,\lambda_{TL}\,\lambda_R\,\tilde{r} \qquad 5.7$$

in which λ_B , λ_{TL} , λ_R and are the correction factors deeply discussed in Sparling (1995) [30], and g is a statistical peak factor (Davenport 1964), which varies between 3.0 and 4.0.

The very simple patch load method had been recommended by the International Association for Shell and Spatial Structures (IASS-WG4) for guyed masts (1981) [32]. In the IASS procedure, the dynamic wind load is applied to the mast using the following load pattern: load acting on all spans simultaneously, load acting on each span individually, and load acting on all but one span in turn. As an example, the required load cases for a two level guyed mast with a

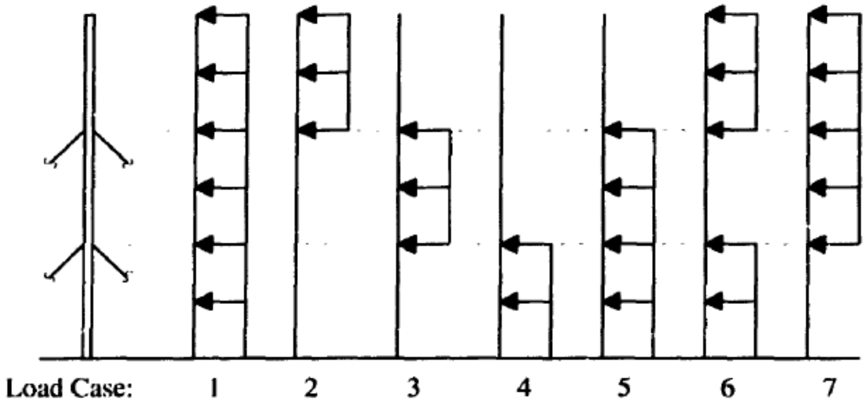

Figure 5-2. Wind load cases for patch load method. [37]

cantilevered top are illustrated in Figure 5-2. The total response envelope for each point along the mast is then obtained by determining the extreme positive and negative values of the dynamic response from the patterned load cases and combining them with the response due to the mean wind load.

Based on the previous method, Sparling (1995) proposed an improved patch load method. The analysis was undertaken in two stages, with the mean wind load effects in a nonlinear approach considered separately from the fluctuating load effects and the peak fluctuating response presented by the effective patch load response in a linear fashion. A series of static load blocks or patches was applied to the guyed mast to simulate the effects of the turbulent wind loading. The response from each load patch was calculated separately and then combined to obtain an estimate of the total dynamic response. Finally, the total response was obtained by adding the response to the mean speed to the response from the dynamic component. There were special considerations for the top cantilevered portion which were presented by Sparling (2007) in [38]. This section, if be present and important, may have a dominating contribution to the intensity of the total displacement of the structure.

A similar approach was employed by Ghafari et al. (2007) [39] and Ghafari and McClure (2008) [40] to calculate the response of cable roof structures to wind loading. According to this study, the total response of the structure under wind loading is obtained from the nonlinear static response under the mean wind speed plus the linear dynamic response to the fluctuating component of the wind.

5.1.3. Dynamic analysis of guyed masts under seismic loads

Meshmesha (2005) [35] concentrated on the study of guyed telecommunication towers under seismic loads. The objective of his research was to develop a finite element model capable of predicting accurately the response of guyed towers when subjected to seismic loading. Moreover, he intended to provide simplified equations to determine the fundamental natural frequencies of guyed masts as well as other response indicators, for instance the base shear, the bending moment, and the axial force in the mast and the tension in the cables. His study included nine guyed telecommunication towers with heights from 60m to 591m [41].

Meshmesha derived two empirical equations to determine the fundamental natural frequencies (in Hz) of guyed masts when the guy modes were not suppressed (Eq. 5.8a), as well as in the case where only the bending frequency of the mast was considered (Eq. 5.8b). The only parameter is the tower height in meters.

$$f_s = 28.5 \; h^{-0.86} \qquad 5.8a$$

$$f_b = 132.4 \; h^{-0.92} \qquad 5.8b$$

In his study, Meshmesha used 24 earthquake records to provide a database for the dynamic response of towers under the different characteristics of earthquake excitation which cover five seismic zones in the United States and Canada with the probability of the exceedance of 2% in 50 years and 10% in 50 years.

Using ABAQUS software, Meshmesha employed the direct integration method in the time domain with a time increment of 0.01s. An interesting feature of his models was the use of the artificial damping option of ABAQUS. Also, he used an equivalent beam column model for the mast and nonlinear cable elements for the guy wires. He also investigated the effect of the antenna weights on the seismic response of these towers and found to be insignificant. Furthermore, he also confirmed that the effect of travel distance of the earthquake along the ground anchors of the tower can not be ignored. Meshmesha also studied the effect of five different bracing configurations, and concluded there was no significant effect of the topology of these bracing configurations on the dynamic and static responses [42]. Finally, he proposed several empirical equations for various guyed mast response indicators. Among them only those related to the Montreal region will be reviewed here, as a sample.

In the case of the shear force and the distribution of the maximum shear forces along the height of the mast for the Montreal area, Meshmesha proposed Equations 5.9, for the probability of exceedance of 2% in 50 years, where in his equation, BS is the maximum base shear in KN, h is the total height of the mast in meters, and V_i / BS is the ratio of shear at the cable attachment points to the maximum mast base shear.

$$BS = 0.0025\, h^{1.76} \qquad h \leq 250\ m \tag{5.9a}$$

$$BS = 0.29\, h \qquad h > 250\ m \tag{5.9b}$$

$$V_i / BS = 1.0 + 6.7\,(h_i/h) - 14.2(h_i/h)^2 + 7.3(h_i/h)^3 \tag{5.9c}$$

Moreover, he continued with equations for the mast vertical reaction and its distribution along the height of the mast for the Montreal area, for the probability of exceedance of both 2% and 4% in 50 years, as in Equations 5.10, where $R_{\max}$ is the maximum mast vertical reaction in KN, and h is the total height of the mast in meters.

$$R_{\max} = 21.1\ h \qquad h \geq 350\ m \tag{5.10a}$$

$$R_{\max} = 2.7^{-3}\ h^{2.5} \qquad h < 350\ m \tag{5.10b}$$

$$P_i / P_{\max} = 1 - 0.71(h_i/h)^2 - 0.32\,(h_i/h) \tag{5.10c}$$

Finally, the maximum bending moment in the mast and the tension magnification factors in the guy cables for the probability of exceedance of 2% in 50 years in Montreal region can be estimated by Equations 5.11, where $M_{\max}$ is the bending moment in the mast in $KN\!-\!m$, h is the total height of mast in meters, and $T_{total} / T_{initial}$ is the ratio of total tension in the cable at a given section to the initial tension.

$$M_{\max} = 0.003\ h^{2.2} \qquad h \leq 250\,m \tag{5.11a}$$

$$M_{\max} = 6.07\ h \qquad h > 250\,m \tag{5.11b}$$

$$M_i / M_{\max} = 5.7(h_i/h) - 10.6(h_i/h)^2 + 5.3(h_i/h)^3 \tag{5.11c}$$

$$T_{Total} / T_{Initial} = 0.12\ (h_i/h)^{-0.132}\ h^{0.453} \tag{5.11d}$$

As mentioned earlier, all these expressions are function of tower height and cannot be extrapolated to other design conditions than those for which they were derived. More comprehensive and rational expressions should involve all the dominant parameters influencing the response. This further suggests the need for a more in-depth study to find simplified methods for the seismic analysis of guyed telecommunication towers.

5.2. Recent seismic research on guyed communication towers in the United States

Due to the presence of active tectonic zones on the West coast of the United States, tower designers have carried out some studies on seismic analysis of communication towers and masts. Fantozzi (2006) [43] studied the nonlinear analysis of a 2000 ft guyed tower, located in California region, with and without mass irregularities. The analysis considered both in-phase and out-of-phase base motion for comparison. The results of the nonlinear analyses were compared to those obtained using the equivalent lateral force method introduced by TIA/EIA-222-G. That was the first time the Standard had seismic loading requirements for towers in regions of high seismicity. His results indicated that the tower met the Code requirements.

Hensley (2005) [44], at Virginia Polytechnic Institute and State University, developed a finite element model of a 120-m tall mast also using ABAQUS. The three-dimensional response of the mast was modeled when subjected to two ground motion records, Northridge, and El Centro, with three orthogonal components. Hensley conducted a parametric study on the dominant structural parameters, and the results were used to characterize the trends in the structural response of guyed masts. He found that for lower amplitude ground motion, when the guying system remained pretensioned, the seismic response of the mast was much more periodic and slower to damp out, suggesting that the behavior was dominated by one of the bending modes. However, when the guys started to exhibit slack behavior, the bending response of the structure drastically changed, and increased moments were observed to be synchronized with the snap loads in the guys. Additionally, dynamic guy tensions were seen to become significantly larger than the initial guy tensions. The parametric study confirmed that cable pretension plays a dominant role on the dynamic response. Conversely, it seemed that the increased bending stiffness of the mast did not have a significant influence. Finally, varying the horizontal input direction of the ground motion had very little effect on the envelope of the dynamic response of the guyed mast.

Finally, with the main objective of developing a systematic evaluation and assessment method, Sullins (2006), at University of Missouri, studied a 45 m guyed tower. His study highlighted the importance of the proper choice of computational software and nonlinear modeling considerations. However his results confirmed that using nonlinear spectrum analysis for guyed masts is tedious and restrictive considering the amount of time and effort to necessary to establish the nonlinear response spectrum for any specific tower under every load case.

5.3. Other international research on guyed masts

A nonlinear dynamic response calculation method was presented by Mossavi Nejad (1996) [45], at University of Westminster, UK, based on step-by-step response calculation in the time domain: equilibrium of dynamic forces at the end of each time increment was established by minimization of the total potential dynamic work. The dynamic loading was generated as a series of cross-correlated earthquake histories defined by the base frequency envelope of typical earthquake records exciting the main frequency components of the structure.

The analysis was applied to the numerical model of a 327 meters tall guyed mast with five levels of stays and a cantilever at the top. The results obtained from nonlinear dynamic analysis were compared to conventional mass-spring system approaches. At the conclusion of his study, Mossavi Nejad mentioned that although the stiffness matrix constructed for the mass-spring model provided good correlation between the static deflections of the guyed mast, the results obtained from the linear dynamic analysis of the mass-spring model are not comprehensive enough to give good comparison with the results of time domain analyses, which highlights the need for more accurate computational models.

In Germany, Zhang and Peil (1996) [46] have employed a cable element model in nonlinear dynamic analysis of guyed towers. They studied the sensitivity of the calculated tower motions to small changes in the boundary and initial conditions. According to Zhang and Peil (1996), it is significant to understand the stability behaviours of guyed towers under earthquake actions, and form the point of view of the structural engineering, the unique parameter for judging the stability of structures is the stiffness. However, it is impossible to compute the stiffness characteristics of guyed towers at every increment step in seismic analysis. As such, the concept of energy increment map was developed to judge the stability behaviours of guyed towers under earthquake loading.

Their results indicated that guyed towers con not be simply classified into the chaotic system although their motion trajectories are sensitive to the small changes of boundaries and initial conditions. The stability behaviours of guyed towers under earthquake actions can be effectively analyzed and judged based on the concept of energy increment map, as it is explained throughout their study.

Appendix A: History of the static and dynamic analysis of guyed masts

Static analysis

Cohen, E. and Perrin, H. (1957), Hull, H.F. (1962), Poskitt, T.J., Livesley, R.K., and Goldstein, A.E. (1963), and Goldberg, J.E. and Meyers, V.J. (1965)

They concentrated on early investigators of the static behaviour of guyed masts by considering the mast as a continuous beam-column resting on nonlinear elastic supports where the spring constants are provided by the lateral stiffness of the guys attached to the shaft.

Cohen, E., and Perrin, H. (1957), "*Design of Multi-level Guyed Towers: Structural Analysis*", ASCE Journal of the Structural Division, Vol. 83, No. ST5, Paper 1356, 29 pp.

Hull, H.F. (1962), "*Stability analysis of multi-level guyed towers*", ASCE Journal of the Structural Division, Vol. 88, No. ST2, pp. 61-80.

Poskitt, T.J., Livesley, R.K., and Goldstein, A.E. (1963), "Discussion: Structural analysis of guyed masts", Institution of Civil Engineering Proceedings, Volume 26, Issue 1, 185 - 186.

Goldberg, J.E. and Meyers, V.J. (1965), "*A study of guyed towers*", ASCE Journal of the Structural Division, ST4.

Goldberg E.J. and Gaunt, T.J. (1973)

They studied the stability of guyed towers using linearized slope-deflection equations to analyze a multi-level guyed tower. They considered the secondary effects due to bending and changes in the axial thrust in the mast based on the small deflection theory.

Goldberg E.J. and Gaunt, T.J. (1973), "*Stability of Guyed Towers*", Journal of the Structural Division, Vol. 99, No. 4, pp. 741-756

Chajes, A. and Chen, W.S. (1979), and Chajes, A. and Ling, D. (1981)

They mainly investigated the stability behaviour of short guyed towers.

Chajes, A. and Chen, W.S. (1979), "*Stability of Guyed Towers*", Journal of the Structural Division, Vol. 105, No. 1, pp. 163-174

Chajes, A. and Ling, D. (1981), "*Post-Buckling Analysis of Guyed Towers*", Journal of the Structural Division, Vol. 107, No. 12, pp. 2313-2324

Schrefler, B.A., Odorizzi, S. and Wood R.D. (1983)

They proposed a method of analysis for combined beam and cable structures using a unified formulation for the geometrically nonlinear analysis of two-dimensional beam and line elements using a total Lagrangian approach.

Schrefler, B.A., Odorizzi, S. and Wood R.D. (1983), "*A total Lagrangian geometrically non-linear analysis of combined beam and cable structures",* Computers and Structures, Volume 17, Issue 1, pp. 115–127.

Odley, E.G. (1966), Williamson, R.A. and Margolin, M.N. (1966), Reichelt, L.K., Brown, M.D., and Melin, W.J. (1971), Rosenthal, F. and Skop R.A. (1980, 1982), and McClure, G. (1984)

These researchers also presented various approaches for static analysis of guyed masts.

Odley, E.G. (1996), "*Analysis of high guyed towers*", ASCE Journal of Structural Division, Volume 92, No. ST1, pp. 169-197.

Williamson, R.A. and Margolin, M.N. (1966), "*Shear effects in design of guyed towers*", Volume 92, pp. 213-233.

Reichelt, L.K., Brown, M.D., and Melin, W.J. (1971), "*Tower: Design System for Guyed Towers*" Journal of the Structural Division, Vol. 97, No. 1, January 1971, pp. 237-251.

Rosenthal, F. and Skop R.A. (1980), "*Guyed towers under arbitrary loads*", ASCE Journal of Structural Division, Volume 106, No. ST3, pp. 679-692.

Rosenthal, F. and Skop, R.A. (1982), "*Method for Analysis for Guyed Towers*" Journal of the Structural Division, Vol. 108, No. 3, pp. 543-558.

McClure, G. (1984), "*Geometric nonlinearities in guyed towers*", Master of science dissertation, Massachusetts institute of technology.

Raman N.V., Surya Kumar G.V., and Sreedhara Rao V.V. (1988)

They considered static analysis using sub-structuring and finite element techniques for large displacement analysis of guyed towers. Two-node 3-D beam-column elements and two-node 3-D truss elements are employed in the finite element model to discretize the mast and the cables, respectively.

Raman N.V., Surya Kumar G.V., and Sreedhara Rao V.V. (1988), "Large displacement analysis of guyed towers", Computers & structures, Vol. 28, Issue, 1, pp. 93-104.

Ekhande, S.G. and Madugula, M.K.S. (1988)

These researchers studied modelling aspects of geometrically nonlinear effects. They presented a three-dimensional nonlinear static analysis of guyed towers consisting of cable, truss and beam member combinations.

Ekhande, S.G. and Madugula, M.K.S. (1988), "*Geometric nonlinear analysis of three dimensional guyed towers*", Computer and Structures, Volume 29, Issue 5, 801-806.

Issa, R.R.A. and Avent, R.R. (1991)

They used a discrete field analysis approach to develop a solution procedure for the analysis of guyed towers. The assumptions of small kinematics and linear elastic behaviour were used for modelling of the tower. The effects of nonlinear cable/tower interaction were also included.

Issa, R.R.A. and Avent, R.R. (1991), "*Microcomputer analysis of guided towers as lattices*", Journal of Structural Engineering, ASCE 117 4 (1991), pp. 1238–1256.

Ben Kahla, N. (1993) and (1995)

He proposed a method for static analysis of guyed towers under wind. An assembly of truss and catenary cable elements was considered in the modelling, and the equivalent beam-column model of the mast was also used.

Ben Kahla, N. (1993), "*Static and dynamic analysis of guyed towers*", Ph.D. thesis, University of Wisconsin-Madison.

Ben Kahla, N. (1995), "*Equivalent beam-column analysis of guyed towers*", Computers & Structures, Volume 55, Issue 4, pp. 631-645.

Fahleson, C. (1995)

The effect of wind and ice loads on guyed masts was studied by Fahleson. Measurements were made on a 323 m guyed TV and radio mast in Northern Sweden, equipped with an extensive data collection system in the winter 1988-89, following the collapse of another mast due to ice overload.

Fahleson, C. (1995), "*Ice and wind loads on guyed masts*". Ph.D. thesis, Luleå University of Technology, SE-97187, Luleå, Sweden, 224 pp.

Dynamic Analysis

Davenport, A.G. (1959)

He developed a linear model to describe the vibration of guy cables under wind loads, assuming that the static deflected shape of the guy is parabolic.

Davenport, A.G. (1959), "*The Wind Induced Vibration of Guyed and Self Supporting Cylindrical Columns*", Transactions, Engineering Institute of Canada, Vol. 3, pp. 119-141.

McCaffrey, R.J. and Hartman, A.J. (1972)

These researchers proposed a mathematical model to predict the dynamic tower response under wind. They analyzed a 302 m tower with fixed base and five guying levels, using truncated modal superposition (the structure was assumed to oscillate linearly about its static equilibrium position). The mast was modelled as an equivalent beam-column with a lumped mass idealization.

McCaffrey, R.J. and Hartman, A.J. (1972), "Dynamics of Guyed Towers", ASCE Journal of the Structural Division, Vol. 98, No. ST6, pp. 1309-1322.

Irvine (1981)

Irvine investigated the dynamic behaviour of guyed towers, with emphasis on analytical expressions for linearized cable vibrations.

Irvine (1981), "*Cable Structures*", The MIT Press, Cambridge, Massachusetts.

Gerstoft, P. and Davenport, A.G. (1986)

They established a simplified procedure to analyze nonlinear guyed towers under wind loading. The guyed mast itself was modelled as a beam on elastic supports.

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Augusti, G., Borri, C., Marradi, L. and Spinelli, P. (1986)

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