Structural Engineering

Seismic Response of Guyed Telecommunication Towers

(Literature Review)

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Gholamreza Ghodrati Amiri

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Introduction

In the wireless, microwave, and satellite communications industry, guyed towers are one of the important structural subsystems. They support a variety of types of antenna systems at great heights to transmit radio, television, and telephone signals over long distances. Guyed towers provide an economical solution for tall towers comparing to self-supporting ones. The main component of these structures is usually a slender trussed steel mast of triangular cross section which is pinned at its base. Sets of inclined guy cables support laterally the mast at several levels along its height. These guy cables are pretensioned and spaced at equal angles around the mast. The various components of a typical guyed tower are shown in Fig. 1.

Very tall towers are essential infrastructures or post-disaster communication systems and therefore, their preservation in the event of a severe earthquake is of high priority. In this regard, the objective of this report is to review previous research on the dynamic behaviour of guyed telecommunication towers in order to apply it in the seismic response studies of such structures. This report will begin with a brief review of the dynamic behaviour of guy cables and then include a general literature review of static analysis of guyed towers. Dynamic analysis and recent studies are presented in more details, and finally, since damping modelling is a complex phenomenon in the geometrically nonlinear dynamic analysis of guyed towers a general review of the topic is included in this relation. In all sections of this report, references are considered in chronological order, and because of the importance of recent research, an individual section is organized by the name of "Survey of Recent Studies".

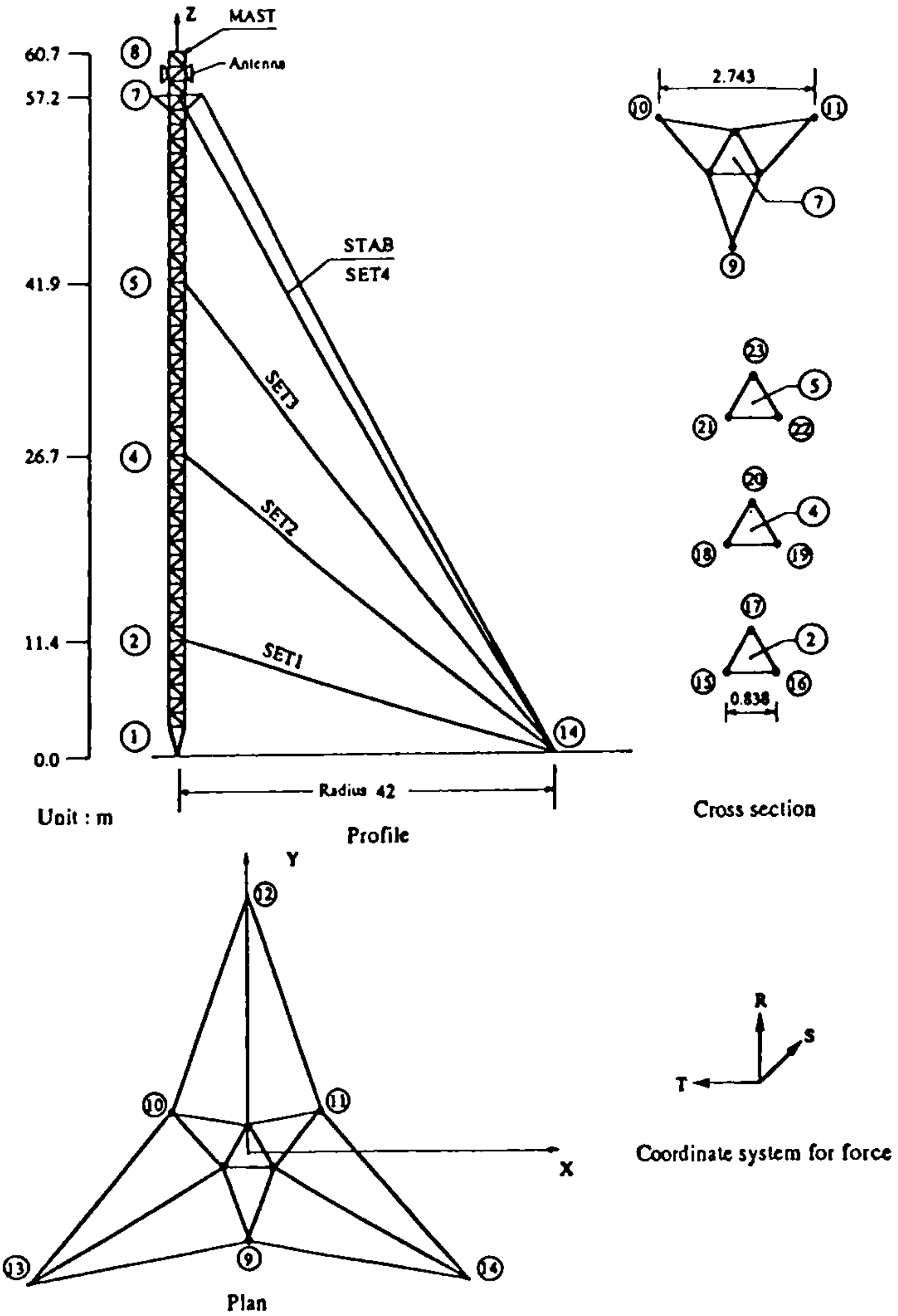

Fig. 1 - Typical guyed tower geometry (Wahba et al. 1992)

Dynamic Behaviour of Guy Cables

The structural behaviour of cables has been studied by a number of researchers. There is a fairly complete description of cable behaviour under several types of loads in Irvine (1981) and Leonard (1988). Irvine (1981) has also investigated the dynamic behaviour of guyed towers, with special interest on analytical expressions for linearized cable vibrations.

The equivalent cable modulus is a commonly used technique to consider the nonlinearity due to sag. Davenport and Steels (1965) have considered this phenomenon to take into account the geometric nonlinearity of the cables. Generally because short guy cables are used at tensions many times greater than the weight of the cable itself, they can be assumed to behave effectively as taut, weightless wires. Therefore, their flexibility under loads could be almost entirely the result of elastic extension of the cable. But in the large cables of tall masts their weight may reach as much as 20% of a safe operating tension in the cable, and it is then no longer taut and sags significantly. The elastic stretch of the cable is only a part of the cable extension under load; the remainder would be the changes of cable-sag geometry. The effect of sag can be taken into account by reducing the guy modulus from the taut wire value. In this paper the theoretical model for the small amplitude, damped vibration of a massive guy cable based on the parabolic approximation to the cable profile is presented. The expressions agree with experiments for moderate amplitude vibrations (Davenport and Steels 1965) under wind loads. Davenport (1959) and Dean (1961) had previously studied various aspects of cable behaviour. Generally, in these papers, guy cables were considered as nonlinear force and moment springs in response to tower displacements, as when lateral loads are applied to the tower, the windward cables are stretched and the leeward cables are slackened.

Huston and Kamman (1982) and Jayaraman and Knudson (1981) contributed to the formulation of a series of numerical methods for cable modelling by using digital computers and the finite element method.

Static Analysis

Most of the early investigations (Cohen and Perrin 1957, Hull 1962, Poskitt and Livesley 1963, and Goldberg and Meyers 1965) studied the static analysis of guyed towers by considering the tower 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. They used solution techniques based on linearized slope-deflection equations.

Shears and Clough (1968) considered a finite element idealisation for an integrated guyed tower analysis in which parabolic cable elements were used for the guys and beam-column elements for the mast. An iterative procedure was used to obtain the nonlinear static response. Aspects regarding stability of guyed towers were not considered by Shears and Clough.

Later, Goldberg and Gaunt (1973) studied stability of guyed towers using linearized slope-deflection equations by analysing a multi-level guyed tower. They considered the secondary effects due to bending and changes in axial thrust of the mast based on small deflection theory. Their result is that instability in guyed towers is not of the bifurcation type, but happens as a relatively large increase in lateral deformation for a small increase in applied loads.

Chajes and Chen (1979) and Chajes and Ling (1981) investigated mainly the behaviour of short guyed towers. Schrefler, Odorizzi and Wood (1983) proposed a method of analysis for combined beam and cable structures. They used a unified formulation for the geometrically nonlinear analysis of two-dimensional beam and line elements using a total Lagrangian approach. The geometrically nonlinear behaviour was considered especially when the cables become slack due to the loss of prestress.

Also, various approaches for static analysis have been presented by Fiesenheiser (1957), Odley (1966), Williamson and Margolin (1966), Reichelt, Brown and Melin (1971), Rosenthal and Skop (1980, 1982), and McClure (1984).

Raman et al. (1988) considered static analysis using substructuring and finite element techniques for large displacement analysis of guyed towers. Two-node (12 degrees of freedom) 3-D beam-column elements and two-node (six d.o.f.'s) 3-D truss elements are employed in the finite element model to discretize the mast and the cables respectively. The equilibrium of guys and mast is solved in the solution technique separately and alternately the compatibility of displacements at the guy support points is applied until the final equilibrium configuration is obtained. Nonlinear effects due to axial load and bowing of the mast, pretension of the cables, and eccentric moments due to cable reactions were considered in the formulation. A linear elastic material behaviour was considered for the mast, and trilinear elasto-plastic behaviour for the guy cables. The guyed tower was studied using both small and large displacement theories (but small strains) for the mast.

Ekhande and Madugula (1988) studied modelling aspects concerning geometrical nonlinear effects. They presented a three-dimensional nonlinear static analysis of guyed towers consisting of cable, truss and beam member combinations. A linear isoparametric formulation for the elements within an updated Lagrangian coordinate framework was employed. Straight line elements with an equivalent reduced modulus of elasticity were used instead of the catenary cables with infinite degrees of freedom . Reduced-order integration and a modified elastic shear modulus in the mast were considered to avoid shear locking in the elements. In addition to the cables, the mast was also considered as a nonlinear element. The tensile strains were considered small.

Issa and Avent (1991) 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 the modelling of the tower. The effects of the nonlinear cable/tower interaction were modelled. The discrete field analysis approach is an alternative to either approximating the space truss as an equivalent continuum beam or repetitive space truss analysis. This alternate procedure is used to obtain closed-form or field solutions to the space truss by discrete field mechanics. This technique was applied to determine the bending stiffness matrix of a truss rather than using equivalent beam properties.

Ben Kahla (1993) has recently proposed a method for the static analysis of guyed towers under wind. An assembly of truss and true catenary cable elements was considered in the modelling. The equivalent beam-column modelling of the mast was also used.

Dynamic Analysis

Many attempts have been made to model the dynamic response of guyed towers. Davenport (1959) developed a linear model to describe the vibration of the guys under wind loads, assuming that the static deflected shape of the guy is parabolic.

McCaffrey and Hartman (1972) also proposed a mathematical model to predict the dynamic response under wind. They analysed a 990 ft tower with fixed base and five guying levels by truncated modal superposition (the structure was assumed to oscillate linearly about its static equilibrium position). The lowest three transverse modes of the guy wires were considered in the response. The mast was modelled as an equivalent beam-column with a lumped mass idealization. They studied the effects of parameters: (1) assuming a parabolic rather than a catenary static deflected shape for the guys; (2)

the number of degrees of freedom of the mast; (3) the ambient temperature; and (4) the higher guy modes, such as, second and third transverse modes, on the free vibration response. They simplified the analysis by assuming that the mast could vibrate only in one plane and that all the guys attached to the mast at a given level have the same dynamic characteristics. Some results of their study are as follows:

(1) The natural frequencies calculated using a parabolic guy model are essentially the same as those obtained with a catenary guy model;

(2) The difference in the two models lies in the calculated mode shapes of the mast corresponding to the various frequencies;

(3) As most of the lower natural frequencies, below 1.27 Hz, are due to the mass inertia of the guys, it is desirable to consider more than just the first mode for each guy in the dynamic analysis;

(4) The even (asymmetric) guy modes may be important in calculating the dynamic response when the slope of the cable is steep or the cable is very heavy.

Very general recommendations were made by the International Association for Shell and Spatial Structures for the seismic analysis of guyed masts, in a special report published by its Working Group 4 on masts and towers (IASS, 1981). This report suggests that a static lateral load proportional to the weight of such structures may be used to model earthquake effects, as is considered in most building design codes for base shear distributions. Designers are then advised to use their national standards for more specific guidelines on dynamic amplification factors and force distribution. A comprehensive nonlinear dynamic analysis of a guyed mast is unlikely to be feasible (IASS, 1981). There are several simplifying assumptions in the IASS report such as considering that the tower oscillates linearly about a given static equilibrium position. It cautions about the use of this assumption in cases where the tension in leeward guys becomes very small and, in cases also when the dynamic displacements of the guy attachment points on the mast become large compared with the displacements of the static equilibrium position. Some recommendations are related to modelling considerations applicable to detailed dynamic analysis for various loads. Simple linear springs are suggested to represent the guys. They are associated with moving masses which properly model the inertia effects of the cables (Fig. 2). The use of a random vibration approach in load modelling and the assumption about the neglecting of wave propagation effects at the ground surface (synchronous ground motion at all supports) are particular recommendations for the seismic analysis. Since seismic loads are already extreme events, their combination with dead loads only is suggested, and it is assumed to occur under still air conditions. It also suggests modal superposition which is valid for linear structures. However, it recommends caution in its use for very tall guyed masts or for unusual towers, which may exhibit significant geometric nonlinearities.

Gerstoft and Davenport (1986) established a simplified procedure to analyze nonlinear guyed towers under wind load. The guyed mast itself was modelled as a beam on elastic supports. These guy supports have nonlinear behaviour for large deflections but linear behaviour was considered for small amplitude dynamic motions. A complicating factor in the treatment of guys is the effect of their mass. This was simplified using the equivalent spring-mass-spring lumped parameter model (Fig. 2b). In dynamic analysis of a taut cable, the first transverse mode is the primary mode of interest. The spring-mass-spring model represents accurately this mode. As the cable slackens, however, contributions to the response from the higher modes increase and the above-mentioned model would be no longer satisfactory. The axial compression in a guyed mast is largely due to the vertical component of the prestressing force in the guy cables. As the axial compression approaches the buckling (critical) load the natural frequencies decrease.

The spring-mass-spring model of guy clusters includes only the contribution of the first transverse mode of vibrations and does not consider torsional vibrations. It does not also consider the additional forces due to guy attachment eccentricities. Karna (1984) improved this spring-mass model to include a viscous dashpot with each spring-mass model (Fig. 2c). His model can be employed in a linear three-dimensional dynamic analysis of guyed masts. The frequency response method and a substructure technique where frequency-dependent springs and dashpots are substituted for the guys were used in this regard. He studied the three-dimensional motion of the guy attachment point to the mast and also the two-dimensional sectional guy motions under the influence of mechanical and aerodynamic damping.

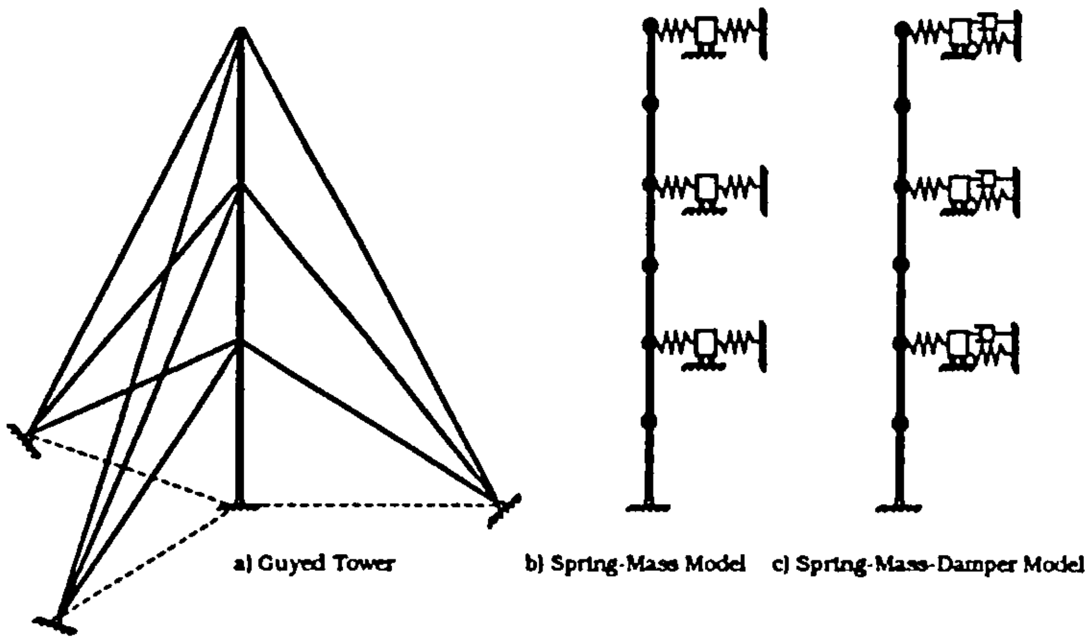

Fig. 2 - Guy Cable Model (Figure from Ben Kahla 1993)

Augusti et al. (1986) modelled a 200 m guyed mast with three guying levels using equivalent linear elastic (Hookean) springs for the guy cables. The spring stiffness depends on the frequency of oscillation; therefore, successive iterations had been used in the calculation of the response to obtain an appropriate stiffness. However, the inertia effects of the cables were not considered and the mast was modelled as a space truss structure with seven lumped masses along its height. The Newmark-β (trapezoidal rule) integration operator was used to calculate the horizontal displacements of the mass points . This formulation is not appropriate for a full dynamic cable-mast interaction.

Buchholdt, Moossavinejad and Iannuzzi (1986) studied time domain methods and compared them with frequency domain methods for structures subjected to wind loads and guy ruptures. They assumed: (a) the deformations of the structural elements remain within their elastic limits and that their materials obey Hooke's law, (b) the guys can be treated as assemblies of linear pin-jointed link elements, (c) structural damping may be expressed as equivalent viscous damping, (d) the dynamic loads may be applied as equivalent concentrated point loads. The mast was assumed to vibrate linearly about the static equilibrium configuration.

Augusti, Borri and Gusella (1990) reported detailed geometrically nonlinear analyses of two guyed towers under wind loading. They analysed a 130 m tall guyed mast with 4 stay levels, and another one, 275 m tall, with 5 stay levels. The mast was modelled by tridimensional beam finite elements. The guy cables were represented by a mesh of five to twelve two-node cable elements. Both types of elements may account for second-order phenomena. Since in the presence of nonlinearities, dynamic analysis can not be carried out in the frequency domain , direct step-by-step integration in the time domain was selected. The implicit Newmark-β integration operator (trapezoidal rule) was used, combined with the classic Newton-Raphson equilibrium iteration procedure at each time step.

Argyris and Mlejnek (1991) analysed a 152.5 m transmitter tower subjected to an idealized sinusoidal earthquake loading (as a rough simulation of an earthquake). Their results indicated that computed displacements have large amplitude, and therefore, serviceability conditions might be exceeded.

Survey of Recent Studies

The geometrically nonlinear seismic response of antenna-supporting guyed towers has recently been investigated by Guevara and McClure (Guevara and McClure (1993), Guevara (1993), McClure and Guevara (1994) and McClure, Guevara and Lin (1993)). They have analysed three towers: 24 m tall with two stay levels (the smallest one), 107 m tall with six stay levels (the intermediate one), and 342 m tall with seven stay levels (the tallest one). A detailed numerical model was employed for the towers. S00E 1940 El Centro and N65E 1966 Parkfield accelerograms were used in the simulations. Each earthquake record was scaled down to match the elastic design spectra of the 1990 National Building Code of Canada for the Montreal region. The combination of lateral and vertical ground motions for the tallest (342 m) tower was studied. Also, the effects of surface wave propagation were considered for the tallest tower by using asynchronous input motions at the ground anchorage points and at the base of the mast.

Guevara and McClure have used ADINA - Automatic Dynamic Incremental Nonlinear Analysis - (ADINA R&D, 1992), a nonlinear dynamic analysis finite element software with direct integration in the time domain to solve the equations of motion. A lumped mass model was employed, and because of the lack of a reliable time-domain damping model for the guy wires, artificial numerical damping was applied instead of structural damping. The small and intermediate masts were modelled as equivalent Timoshenko beam-columns, whereas a detailed three-dimensional truss model was used for the tallest mast. Tension-only three-node isoparametric truss elements with initial prestress were used to model the guy cables. In order to account for geometric nonlinearities, a large kinematics (but small strains) formulation was used for the cable model.

Their results indicated that the high frequency components of the excitation effects the shortest tower. However the magnitude of the peak response was not considerable because of the low seismicity level of the Montreal region. They have found more important dynamic amplifications in the extreme guy clusters (top and bottom) for the response of the two other towers. The maximum values of the mast shear were occurred at intermediate elevations in their studies. They have used the Large Mass Method (Léger et al., 1990) to study the effect of asynchronous base motion and have found significant effects only in the guy wire tensions of the bottom cluster. From the point of view of reducing the required total analysis time, the use of the equivalent model formulation for the tall mast is not effective due to the large number of member properties along the height of the mast. Furthermore, the detailed structure exhibits a complex torsional behaviour that cannot be reproduced in the equivalent beam-column formulation. Therefore, detailed modelling of tall masts would be preferable to equivalent beam-columns. Also, their results indicated that the cable-mast interactions are significant in the frequency range of the lower axial modes of the mast. They have found some important dynamic interactions between the mast and the guy wires by combining vertical and horizontal ground accelerations.

A numerical study of the transient dynamic response of guyed telecommunication towers subjected to sudden ice shedding from the guy wires has been carried out by Lin (1993) and reported in McClure and Lin (1994) and McClure, Guevara and Lin (1993). The ADINA commercial software has been used for detailed nonlinear dynamic analyses. They have analyzed three towers with heights of 24.4, 60.7 and 213.4 m, and with two, four and seven stay levels respectively. In one case, artificial numerical damping was employed to compare with the undamped results. They have used direct integration in the time domain. The beam-column elements were used to model the mast with stiffness properties equivalent to those of the three-dimensional lattice structure. They have modelled the guy wires by tension-only three-node isoparametric truss elements with initial prestress. In order to account for geometric nonlinearities, a large kinematics (but small strains) formulation was used for the cable model.

Referring to the Canadian Standards Association in CAN/CSA-S37-94 (Draft June 1994) for structural design of antenna-supporting structures:

4.11 Earthquake Effects (E)

The effects of earthquake are not covered by this Standard. In most cases earthquake effects on towers are less than the effects due to wind, but they should be considered for susceptible towers of critical importance (e.g. post-disaster communication systems) in high earthquake zones.

Note: *See appendix L.*

Appendix L - Seismic Analysis of Towers

Note: *This Appendix is not a mandatory part of this Standard.*

L4. Seismic Analysis of Guyed Towers

Recent numerical studies reported by Guevara (1993) and Guevara and McClure (1993) have indicated that if one wishes to perform a detailed dynamic analysis of a guyed tower, modelling of the structure must allow for geometric nonlinearities and potential interactions between the mast and the guy wires. These interactions can be properly assessed only if inertia properties of both the mast and the guy wires are correctly modelled. The simplified model of a continuous beam on elastic supports, which is still used by some designers to carry out static analyses, is therefore not appropriate. The mast itself is relatively lightweight (wind forces will be greater than gravity forces) and since its mass is more or less linearly distributed over its height, the lateral inertia forces generated by seismic excitations of this distributed mass are not going to be as significant as the wind forces. The most important seismic effect appears to be induced by cable-mast interactions, as transverse cable vibrations induce a vertical dynamic force in the mast that may excite its lowest axial modes and, as a result, may create significant vertical forces. These effects are amplified when vertical input accelerations are combined with the usual horizontal accelerations; the vertical effects induced in the mast also propagate into the guy wires and generate additional amplifications in the cable tensions. Numerical studies have shown that dynamic amplifications in the guy wire tensions are more likely to be significant in the top and bottom clusters of multi-level guyed towers.

Detailed nonlinear seismic analyses are far more complex than response spectrum analyses, and not always necessary. A frequency analysis, as suggested in Section L3, for the initial configuration can help to identify the sensitive frequency range of the tower and potential interaction effects due to clustered frequencies. This information will help the designer decide whether it is necessary to proceed with a more detailed nonlinear dynamic analysis.

Dynamic analysis of guyed towers under wind loading has been studied recently by Ben Kahla (1993, 1994). An equivalent beam-column model has been used for the guyed tower. He has used the lumped mass method for the mast. He has assumed cable elements are perfectly flexible with uniform cross-section between their attachment points. The exact mathematical model of an elastic catenary was employed for the formulation of these cable elements. This model is subjected to arbitrary combinations of wind, ice and dead weight. A fictitious linear viscous damper with 5% critical damping in parallel with each cable element was applied in the model. The energy dissipation in the material and the friction due to the inner-strand rubbing can be modelled with these damping elements. In addition the aerodynamic damping of guy cables can be reproduced by these dampers. Because in moderate and high winds, aerodynamic damping is significantly greater than structural damping, aerodynamic damping was modelled and structural damping was neglected. The NSDAGT program (Ben Kahla, 1993), a dynamic analysis finite element software, has been used in this research. Two towers were analysed using a two-dimensional model, both are 146.3 m (480 ft) tall towers with one and three stay levels respectively.

Ben Kahla's results have shown that large amplitude oscillations of guy cables are possible. These large oscillations are accompanied by large guy tensions that would likely result in the failure and collapse of the structure. These failures could be by breakage of one or several guy cables or by local buckling of some of the mast elements. Also, the lowest cable modes of vibration are not the critical frequencies. This observation implies that the spring-mass model could be invalid to represent guy cables in transient analysis, since the only fundamental mode of a guy vibration can be represented by this model. Because of the potentially severe consequences of the large amplitude oscillations of guy cables, Ben Kahla recommends detailed dynamic analysis for guyed structures under wind loads. According to Ben Kahla (1993), the Australian code (AS3995(Int)-1991, 1991) for the design of steel lattice towers and masts suggests full dynamic analysis under wind loads for those towers and masts with a fundamental frequency less than 1 Hz.

The modelling of tall guyed towers has also been recently studied by Gantes et al. (1993), in relation to an investigation on the collapse of a 579 m (1900 ft) tall guyed tower under ice and wind loads. Based on their investigation, some structural analysis recommendations relating to loading and modelling concerns were proposed. Their results have shown that an equivalent beam model would be a simple and acceptable solution for the mast, while equivalent springs are satisfactory for cable modelling for preliminary analysis. A nonlinear truss representation in the sagged configuration was suggested for a more exact finite element analysis when cable elements are not available.

Modelling of Damping

The damping factor is a very important variable in determining the maximum amplitudes of vibration and the associated stresses in a guyed tower. The damping forces in a guyed tower arise mainly from two sources. The first source is structural (material or hysteretic) damping. This damping is partly due to frictional damping within the twisted strands, and partly to structural damping in the connections of mast. The other source of damping is aerodynamic from the viscosity of air. The aerodynamic damping is affected significantly by the cable motion (IASS, 1981).

Structural damping of a guyed tower includes the internal damping of the guy cables and mast. The first source, usually the most significant of the two, is the internal rubbing of the strands of the cable during any flexing action or transverse vibrations. Solid and straight-strand steel cables exhibit little structural damping, but it is increased considerably in twisted-strand cables, as they exhibit large hysteresis in their oscillation. Because of the geometric nonlinearities in the cables, their frictional damping is both frequency dependent and nonlinear with respect to the amplitude of the motion and also varies with the nature of motion (transverse, axial, or both), which causes damping modelling to be very difficult in the time domain. A.-T. Yu (1952) had done a few experiments on lighter cables (0.375 in. and 0.279 in. diameters, carbon steel strand) used in power transmission and had shown that the equivalent viscous damping from this source is not more than 5% of critical equivalent viscous damping in the 0.279 in. cable and 3% in the 0.375 in. one. Damping in the lattice mast depends on the material and the form of construction (welded, riveted, bolted). Bolted structures provide more structural damping compared to the welded ones because of the energy dissipation at the bolted joints. In the case of fully welded structures, the International Association for Shell and Spatial Structures (IASS, 1981) recommends equivalent damping of 1.2% of the critical viscous damping for the mast, 2% for high strength friction bolted steelwork, and 3% for normal bolted and riveted steelwork. The connections of cable-to-mast, cable-to-ground, and mast-to-ground have also some frictional effects as a source of structural damping. According to Kennedy et al. (1980), Sachs (1972) proposed that the variation of the structural damping factor or damping ratio ζ is from about 0.003 to 0.03. Nakamoto and Chiu (1985) suggested that the critical damping ratios are of approximately 2-5% for frequencies between 0 and 0.5 Hz, and approximately 1-2% for 0.5-5 Hz. There is also some damping effect from tower attachments (transmission lines, ladders, platforms, antenna mounts).

Due to the difficulties associated with realistic time-domain modelling of damping (structural and aerodynamic) for nonlinear analysis, Guevara (1993) has used artificial numerical damping instead of structural damping. In that study, numerical damping was also employed to filter out numerically generated high frequency components, recognizing that in reality spurious high frequency components would likely be quickly filtered out by physical damping (structural damping). However, numerical damping can not be calibrated with physical damping in nonlinear multiple-degree-of-freedom systems.

Algorithmic damping (numerical damping) can be generated by direct integration operators (e.g. Newmark-β operator). To generate numerical damping with the Newmark-β integration operator, Guevara (1993) tested two sets of values of parameters. The first set was $\delta=0.6$ and $\alpha=0.3025$ which reduced displacement amplitudes by more than 35% with respect to the trapezoidal rule solution. Because this reduction was excessive, a second set of parameters with $\delta=0.55$ and $\alpha=0.3$ was studied. This combination proved sufficient to eliminate the spurious high frequency components. However, artificial damping is not an ideal substitute for the cables and mast damping. Especially in very tall towers for which the natural frequencies are relatively lower, algorithmic damping would not be sufficient to filter out the response peaks.

Ben Kahla (1993, 1994) has used a fictitious linear viscous damper in parallel with each cable element instead of the structural and aerodynamic damping. Because in moderate and high winds, aerodynamic damping is significantly greater than structural damping, therefore the aerodynamic damping was modelled and the other one was neglected. He has used 5% critical damping for guy parallel dampers. Therefore, further research is still needed to assess the damping characteristics of cables and mast, and to develop an appropriate damping model for finite element nonlinear analysis in the time domain.

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