Vertical seismic behaviour of tall guyed telecommunication towers

Ghodrati Amiri, Gholamreza[1]

ABSTRACT

Guyed towers are commonly used for communication structures, and support radio, television, and telephone broadcasting antennas, or are themselves radiators to transmit communication signals. Very tall towers are a fundamental component of post-disaster communication systems and in the category of lifeline and infrastructure, therefore, their protection during a severe earthquake is of high priority and accordingly their seismic performance should be properly evaluated.

Since the two different components of guyed towers (cables and mast) have different dynamic performance, their interaction can be very important. One of the important aspects of this interaction is the vertical response of tall guyed towers, which include the vertical component of cable response on the mast. No simple method has yet been developed to estimate the vertical response of these structures and these effects are usually ignored in the design. In this paper, the vertical seismic behaviour of eight real guyed telecommunication towers with heights varying from 150 to 607 m, is investigated. This study is based on detailed nonlinear seismic analysis. Each tower was subjected to three different classical seismic excitations (El Centro, Parkfield and Taft earthquakes) for the seismicity level of the Victoria region, which has one of the highest seismicity levels in Canada.

INTRODUCTION

The recent developments in the telecommunications industry have led to an extensive use of tall guyed towers in these systems. They are needed to support a variety of antenna systems at great heights to transmit communication signals over long distances. Figure 1 shows a typical geometry of tall guyed tower. In this study, eight real guyed telecommunication towers were selected for detailed modelling. These towers represent different heights, number of guying levels and locations. In practice, those guyed towers taller than 150 m usually provide economical solutions comparing to self-supporting towers. Therefore, the lower height limitation for tall towers could be 150 m, which is a common criterion to classify towers with respect to their heights. In this regard, those available guyed towers taller than 150 m were selected for the simulations. These towers are listed in Table 1. It should be mentioned that the 607-m tower is located in Sacramento, California, and is one of the tallest guyed telecommunication towers in North America.

MODELLING CONSIDERATIONS

The nonlinear finite element software ADINA (Automatic Dynamic Incremental Nonlinear Analysis) is used in this research (ADINA 1992). A detailed three-dimensional truss model is employed for the mast where all elements resist only axial forces. Guy cables are modelled with three-node truss elements. The stress-strain law is defined only in tension to allow for cable slackening effects to be modelled during the earthquake vibrations. A large kinematic formulation (but small strains) is used for the mast and the cable stiffness to account for geometric nonlinearities. The lumped mass formulation is employed in the analysis, and material properties are assumed linear elastic. An equivalent viscous damper (2% of critical viscous damping) is used in parallel with each element to model structural damping. Since earthquake loads are assumed to occur under still air conditions (IASS 1981), aerodynamic damping has not been modelled.

The nonlinear dynamic analysis is done by direct step-by-step integration in the time domain. The numerical integration procedure selected is the Newmark-β method (with γ=0.5 and β=0.25, i.e. the constant average acceleration method or trapezoidal rule). The BFGS (Broyden-Fletcher-Goldfarb-Shanno) equilibrium iteration procedure is employed in combination with the Newmark-β method to solve nonlinear equations, which is very effective compared to the other iteration procedures available, and stiffness matrix updates are performed at every time step. Also, the energy convergence

[1] Assistant Professor, Department of Civil Engineering, Iran University of Science & Technology, P .O. Box 16765-163, Narmak, Tehran 16844, Iran, Tel: (98-21)789-6623, Fax: (98-21)745-4053, e-mail: ghodrati@sun.iust.ac.ir.

criterion is used to bound the iteration process. The subspace-iteration procedure is used in the frequency analysis.

EARTHQUAKE ACCELEROGRAMS

In this study, three classical horizontal earthquake accelerograms have been selected for use in the numerical simulations, representing different types of earthquake loading (Fig. 2). The first one is the S00E 1940 El Centro earthquake containing a wide range of frequencies and several episodes of strong ground motion; the second one is the N65E 1966 Parkfield earthquake representing a single pulse loading with dominant lower frequencies; and the third one is the S69E 1952 Taft earthquake with high frequency content and strong shaking with long duration. These earthquakes are selected to reflect realistic frequency contents as exhibited by real ground motions. It should be noted that the earthquake direction was selected to coincide with the principal direction of the mast cross section, as indicated in Fig. 1, to create maximum seismic effects in bending.

The earthquake records were scaled to fit as much as possible the elastic design spectra of the 1995 National Building Code of Canada (NBCC) for the Victoria region (Peak Horizontal Ground Acceleration = 0.34g and Peak Horizontal Ground Velocity = 0.29 m/s) which has one of the highest seismicity levels in Canada. The scaling allows the comparison of the response of the tower for different accelerograms with the same intensity. Schiff's scaling procedure (Schiff 1988) is used with scaling factors of 1.19 for El Centro, 0.69 for Parkfield, and 2.68 for Taft accelerograms. Referring to the National Building Code of Canada 1995 (Commentary J), and considering that the ratio of vertical-to-horizontal accelerations depends on site conditions and varies widely, an average range of 2/3 to 3/4 is proposed for this ratio. In this study, in the absence of real data on vertical accelerograms, a ratio of 3/4 is assumed.

RESULTS AND DISCUSSIONS

Seismic component of mast axial force at the base

The maximum dynamic component of the axial force at the base of the mast is investigated as a percentage of the total weights of the tower. Since the axial effects are more important when the vertical component of the earthquake is also considered, the case of combined vertical and horizontal accelerations is studied. It is observed that for guyed towers with usual initial cable tension (i.e. around 10% of their ultimate tensile strength), the maximum dynamic component of the axial force at the base of the mast due to combined vertical and horizontal earthquake motions, is about 80% of their total weight. From the point of view of the strength and stability of the mast, these guyed towers may be sensitive to seismic vertical effects. The maximum contribution of the mass of the cables to the dynamic component of the axial force at the base of the mast, due to combined vertical and horizontal earthquake motions, is 10% of the total weight of the tower which is small compared to that of the mast. It should be mentioned that the mass of the mast accounts for 69% to 77% of the total tower weight, leaving 23% to 31% to the guy cables. However, tower attachments such as antennas and accessories (e.g. ladder, transmission lines, lights for aircraft warning, etc.) are not included in the total weights. The variation of the mass of the mast along the height of the towers is almost uniform for most towers. Exceptions are due to the presence of torsional resistors (or outriggers) in the towers. In the 607-m tower the mast becomes lighter in the top portion around the top three guy stay levels. It is to be mentioned that for most cases the initial cable tension varies from 8% to 12% of the UTS (ultimate tensile strength) with 10% being the most common practice.

Distribution of maximum dynamic component of mast axial forces with tower elevation

The distribution of the maximum dynamic component of the axial forces in the mast along the height for all eight towers is summarized in Fig. 3. The response to combined horizontal and vertical earthquake motions is considered here. In this graph the horizontal axis is representing the ratio of the maximum dynamic component of mast axial forces to the maximum dynamic component of mast base axial force. This ratio is expressed as a percentage. The vertical axis is showing the ratio of the sectional elevation of mast to the total tower height. It can be seen in Fig. 3 that the results of all the towers are relatively close to each other and form a narrow band. They can be appropriately (and conservatively) represented by a parabolic curve fit (darker line in Fig. 3). The parabola has the following equation:

$$(P_{dyn}/\text{Max B.A.}) = 100 - 95\,(h/H)^2 \qquad (1)$$

where $(P_{dyn}/\text{Max B.A.})$ is the percentage ratio of the maximum dynamic component of axial force in the mast at a section of given elevation (h) to the maximum dynamic component of the axial force at the base of the mast. It is noted that H is the total height of the tower.

CONCLUSIONS

The results are obtained for the seismicity level of the Victoria region in Canada with PGA = 0.34g and PGV = 0.29 m/s. The horizontal accelerograms with 75% of their amplitude are used for the vertical earthquake. The seismic zone selected is one of the most severe ones in Canada. The height of the guyed towers studied in this research varied in the range of 150 to 607 m. Since these guyed towers exist and are typical of other towers, the observations can be generalized. A summary of the important results in this paper are:

I) Some results obtained from this study have shown that as an upper bond estimate to the maximum dynamic component of the axial force at the base of the mast is in order of 80% of the tower weight for guyed towers with usual initial cable tension (i.e. around 10% of their ultimate tensile strength).

II) The maximum contribution of the mass of the cables to the dynamic component of the axial force at the base of the mast, due to combined vertical and horizontal earthquake motions, is 10% of the total weight of the tower weight for guyed towers with usual initial cable tension (i.e. around 10% of their ultimate tensile strength).

III) A parabolic curve fit (Equation 1) is proposed for the distribution of the maximum dynamic component of the axial force in the mast along the height of the tower due to combined vertical and horizontal earthquake motions.

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NOTATION

B.A. = Dynamic component of mast base axial force

g = Gravity acceleration

H = Tower height

h = Sectional elevation of mast

P_{dyn} = Maximum dynamic component of axial force in the mast at a section of given elevation

PGA = Peak horizontal ground acceleration

PGV = Peak horizontal ground velocity

β = Second parameter of Newmark-β method

γ = First parameter of Newmark-β method

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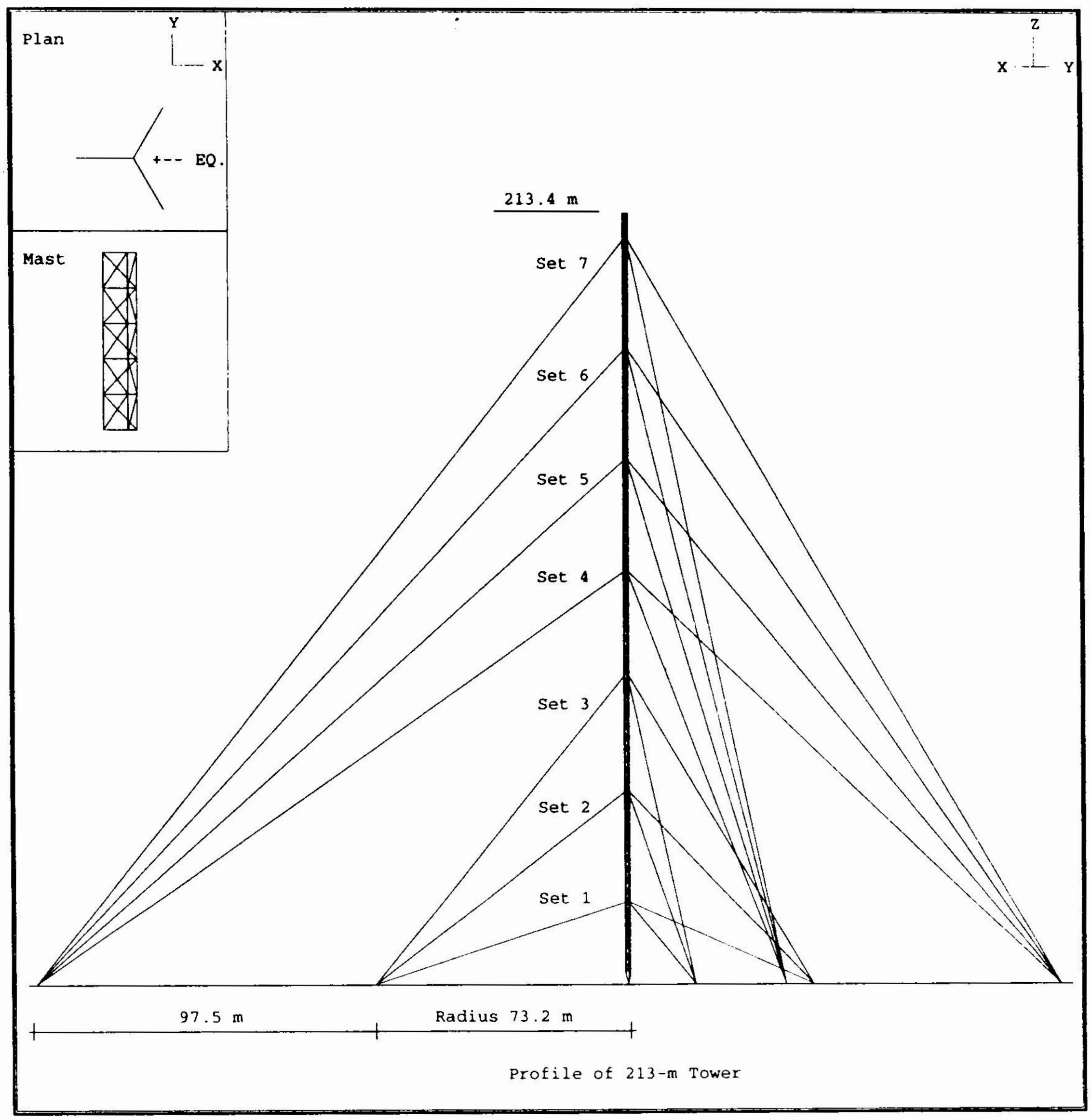

Fig. 1. Typical geometry of tall guyed tower

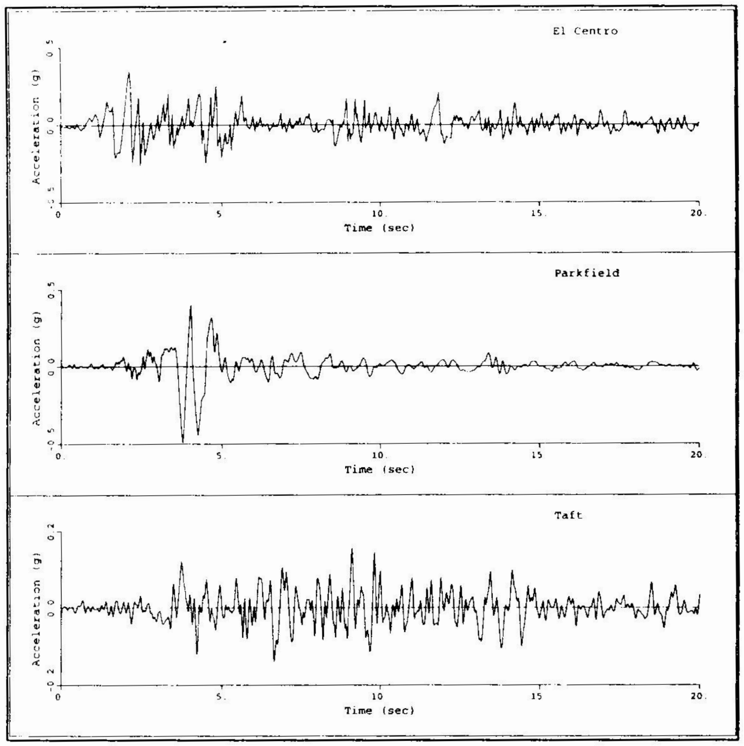

Fig. 2. Earthquake accelerograms

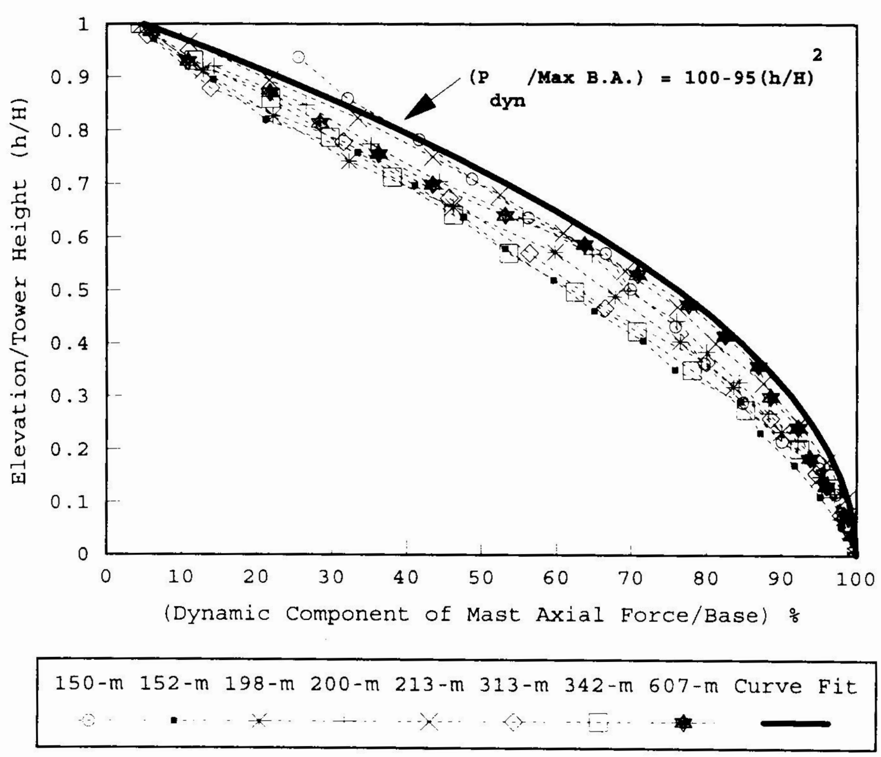

Fig. 3. Distribution of maximum dynamic component of mast axial forces along height (horizontal + vertical)

Table 1. Guyed towers used in numerical simulations

Tower height (m)	Number of guying stay levels	Number of anchor groups	Number of outriggers	Panel width (m)	Panel height (m)	Location (Source)
607.1	9	3	0	3	2.250	USA., California, Sacramento (LeBLANC & Royle Telcom Inc.)
342.2	7	2	1	2	1.524	Canada (Wahba et al. 1992)
313.9	5	2	0	2.140	1.524	Canada (Wahba et al. 1992)
213.4	7	2	0	1.524	1.524	Canada (Wahba et al. 1992)
200	8	3	0	1.800	1	Argentina, Buenos Aires (Estudio Ing. M. Oberlander)
198.1	6	2	1	2.134	1.524	Canada, Prince Edward Island, Charlottetown (Trylon Manufacturing Co. Ltd.)
152.4	8	2	2	0.838	0.610	Canada, Alberta, Elk River (AGT, LeBLANC & Royle Telcom Inc.)
150	7	2	3	1.300	1	Canada, Alberta, Little Buffalo (LeBLANC & Royle Telcom Inc., AGT)

Note: All the towers have a triangular mast, except that of the 200-m tower which is square.