# Validation of a Linearized Seismic Analysis Method for Tall Telecommunication Masts 

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#### Abstract

Ghafari and McClure ( 2011,2012 ) have developed a new robust linearized seismic analysis method for tall guyed telecommunication masts for design check purposes. This method provides a conservative prediction of the maximum lateral seismic displacements of the mast at the multiple stay levels. Its analytical development duly accounted for the geometric nonlinear response of the guy cables and the cable-mast dynamic interactions. The method was calibrated using detailed nonlinear dynamic analysis of nine real guyed masts subjected to a few earthquake signatures. This paper presents a more extensive evaluation of the proposed method and discusses some trends observed in the results for two selected guyed telecommunication masts under the effects of eighty-one recorded Californian earthquakes. Based on the maximum mast displacements predicted by the linearized method, approximate values are calculated for the maximum cable reactions at their tower attachment point, the maximum cable tensions, and the internal bending moments and horizontal shear forces in the mast. A comparison of the detailed nonlinear dynamic analysis results with those obtained from the linearized simplified method confirms the reliable performance of the simplified method.


Keywords- Seismic Design; Telecommunication Masts; Guyed Towers; Ambient Vibration Measurement; Structural Dynamic

## I. INTRODUCTION

A new robust linearized seismic analysis method for guyed telecommunication masts was presented in Ghafari and McClure (2012). Following a detailed study of the geometric nonlinear dynamic response of individual guy cables (Ghafari and McClure, 2009 and 2011), a condensed dynamic model of the mast was created that captured its horizontal seismic response. The condensed stiffness and mass matrices of nine real telecommunication masts were determined and their seismic response was calculated for five different earthquake signatures. The results indicated appropriate and safe predictive performance of the proposed method in view of approximate seismic design checks, i.e. those tall masts that would be flagged as critical by the approximate procedure and would require a more detailed seismic analysis. No such procedure has been introduced yet in North American design codes (EIA/TIA-222-G 2006, CAN/CSA S37-13).

This paper presents a more extensive validation of the proposed method with the study of two guyed telecommunication masts under the effects of eighty-one recorded Californian earthquakes. The masts selected are a $342-\mathrm{m}$ tower, representative of the height range of applicability of the proposed method ( 180 to 350 m ), and a $607-\mathrm{m}$ tower, representative of an extreme height case that should be treated with detailed nonlinear dynamic analysis. Based on the maximum mast displacements at guying levels obtained from the linearized condensed analysis method, the maximum resultant forces exerted by the guy cables at their mast attachment points and the maximum cable tension forces are estimated. The envelopes of the bending moments and shear forces in the mast are also obtained; an approximate value of the mast axial force can also be calculated which does not include the effects of vertical accelerations.

## II. MAST CASE STUDIES

The two masts selected as case studies have been previously studied in detail by Amiri (1997), Faridafshin and McClure (2008) and Ghafari and McClure (2009, 2011 and 2012). Fig. 1 shows global elevation and plan views of the finite element


Fig. 1 Geometric finite element mesh layout of (a) 342-m mast and (b) 607-m mast
meshes created for the two structures. The 342.2-m mast (Fig. 1a) comprises seven stay levels arranged in two ground anchor groups for a total of 24 guy wires. The dimensions of the mast panel width and height are 2.00 and 1.54 m , respectively, and there is a torsional stabilizer (outrigger) at the second stay level from the base. The tower is located in Canada. The second tower is $607.1-\mathrm{m}$ tall and comprises nine stay levels arranged in three ground anchor groups for a total of 27 guy wires (Fig. $1 \mathrm{~b})$. The dimensions of the mast panel width and height are 3.00 and 2.25 m , respectively. This tower is located in California, United States.

## III. SEISMIC LOADING

Eighty-one Californian earthquake records of horizontal accelerations, starting from the Humbolt Bay earthquake in 1937 to the Northridge earthquake in 1994 and representing different frequency contents and strong motion durations have been considered in this study. All data have been taken from the PEER (Pacific Earthquake Engineering Research Center) Strong Motion Database. Detailed information on the selected earthquake records and the corresponding seismic analysis results are presented in Ghafari (2010).

Since most earthquake records of the PEER database are available for several seismological stations, the record with the maximum peak ground acceleration (PGA) has been selected for analysis. Baseline correction was performed using SeismoSignal software (Seismosoft 2010) and all selected records were scaled to have the same peak horizontal ground acceleration value (PGA) of 0.3 g . This simple scaling corresponds to the PGA value compatible with the uniform hazard elastic design spectrum of the Canadian National Building Code (NRC 2005) for Victoria (British Columbia), specified for stiff ground and a probability of exceedance of $2 \%$ in 50 years. The PGA scaling is meant to facilitate the comparison of the results obtained from all the records analysed and is not used for seismic vulnerability assessment of the structures, which would require due consideration of seismic input and soil conditions at the tower site.

The frequency content (discrete Fourier transform and power spectral density) of the individual records was studied. For example, Fig. 2 illustrates the horizontal motion time histories of the Northridge 1994 record with the corresponding frequency content diagrams of the acceleration record. Typical high-energy frequency content of world earthquakes is known to lie in the $0.1-10 \mathrm{~Hz}$ range. The average frequency content of the selected 81 horizontal earthquake records is represented in Fig. 3, showing peak energy in the $0.5-4.0 \mathrm{~Hz}$ range. Bracketed durations of recorded strong motion (with horizontal acceleration in excess of 0.05 g ) typically vary between 15 s and 30 s , but a conservative value of 40 s was used in analysis. Other modeling considerations related to detailed nonlinear seismic analysis are presented in Ghafari and McClure (2011 and 2012).

(a)

(b)



Fig. 217 Jan 1994 Northridge earthquake 24207, Pacoima Dam station (CDMG): (a) acceleration, (b) velocity and (c) displacement time histories; (d) discrete Fourier transform amplitude and (e) power spectral density


Fig. 3 The average discrete Fourier transforms amplitude of the selected 81 Californian earthquake horizontal accelerograms

## IV. RESULTS AND DISCUSSION

## A. Mast Displacements and Cable Tensions

The detailed numerical models of the two structures were analyzed in ADINA (2004) under the effects of the 81 earthquake signatures prescribed as synchronous horizontal displacements at the guy cable ground support points and at the base of the
mast. Time history plots of the horizontal displacements and cable tensions at the middle point of each guy cable were examined. Profiles of the maximum horizontal displacements of the mast at cable stay levels were also extracted from the detailed finite element analysis results for comparison with the displacement results obtained from the condensed seismic analysis method. This is discussed next and illustrated in Fig. 4 to 7. Detailed results of the complete study are available on line in Ghafari (2010).

Fig. 4 illustrates masts displacements at different cluster attachment points of studied tower under the Northridge record as an example.


Fig. 4 Masts displacements at different cluster attachment points: (a) 6th cluster of 342-m mast and (b) 9th cluster of $607-\mathrm{m}$ mast (clusters are numbered from bottom)


Fig. 5 Maximum horizontal displacements of the $342-\mathrm{m}$ mast - average response under the effects of 81 earthquake records scaled to $\mathrm{PGA}=0.3 \mathrm{~g}$ and statistical distribution

The results labelled as "Simplified numerical models (ADINA)" in Fig. 5 to 7 are those obtained by replacing the detailed finite element models of the guy cables by the equivalent linearized springs while the mast structure is the same detailed lattice configuration in both numerical models. The two other curves were obtained from the condensed dynamic analysis models, based on the linearized procedure introduced in Ghafari and McClure (2012). In this procedure, the condensed (non-symmetric) flexibility matrix of the mast is assembled (the corresponding stiffness matrix is also non symmetric) and the corresponding results are labelled as "Analytical method (Non symmetric stiffness matrix)". A last simplification is introduced where the resulting stiffness matrix is made symmetric by averaging its off-diagonal terms, and the corresponding results are labelled as "Analytical method (Symmetric stiffness matrix)". The purpose of this comparison is to illustrate and assess whether the simpler, symmetric condensed stiffness approach is satisfactory for approximate seismic analysis and design checks.

Fig. 5 plots the envelope curves of the average displacements simulated for the $342-\mathrm{m}$ tower under the 81 earthquake records. Inserts in this Figure represent the statistical distribution of the results obtained using the condensed analytical method with symmetric stiffness matrix, while the line shows the theoretical Gaussian distribution. Fig. 6 shows the envelope curves of the average cable tension forces calculated for this tower under the 81 earthquake records. In this Figure, the two sets of results identified with "Analytical Method" were derived based on the linearized spring forces representing the effects of the guy cables under the mast deformation patterns reported in Fig. 5. Cable tensions are reported at their corresponding stay level.

The purpose of the condensed linearized method is to provide a conservative, over prediction of the mast displacements at stay levels for approximate design checks. As shown in Fig. 5, this is clearly achieved by the condensed symmetric matrix method at all stay levels except the bottom one. Fig. 6 confirms this conservative predictive trend in all cable tensions (including the bottom cluster) for the method based on the condensed symmetric stiffness matrix of the mast. The nonsymmetric method provides less conservative (and more accurate) estimates of the mast displacements than the symmetric method, but it tends to underestimate the maximum response at the top cable clusters, which is not conservative.

The statistical distribution of the results obtained for the 81 base motion simulations is shown in Fig. 5 for two selected elevations. In particular, the narrow distribution of the results at the higher cluster confirms that the average value provides a reliable estimate of the maximum probable response of the structure.

Although the condensed linearized method is restricted to tower heights not exceeding 350 m , a global comparison of the different methods is presented in Fig. 7 for the $607-\mathrm{m}$ mast. These results further confirm that the condensed analytical method with symmetric stiffness matrix provides conservative results of the maximum sway displacements of the mast.

Overall, the results indicate that a more accurate prediction of the mast displacements and cable tension forces is obtained using the condensed model with non-symmetric stiffness matrix. However, the simpler approximate symmetric stiffness matrix is more appropriate because it provides a safe overestimation of the mast displacements and cable tension forces. This is the method recommended for rapid seismic design checks.


Fig. 6 Maximum cable tension forces of the 342-m mast average response


Fig. 7 Horizontal displacements of the $607-\mathrm{m}$ mast - average response under the effects of 81 earthquake records scaled to PGA $=0.3 \mathrm{~g}$

## B. Internal Forces in the Mast

In addition to antenna serviceability checks based on the mast deformation profile, telecommunication tower design requires strength verifications for the guy cable tensions (and ground anchor reactions) and other seismic force response indicators such as the axial compression, the maximum shear force at each guy cluster attachment level and the maximum bending moment along the mast. The maximum value of the cable resultant force can be approximated using the predicted value of the maximum horizontal displacements in the mast, $\Delta$ and the equivalent dynamic stiffness of the guy cables introduced in Equation 12 of Ghafari and McClure (2012). Accordingly, the horizontal component of the resultant force, $\mathrm{N}_{\mathrm{h}}$, can be approximated by Equation (1) and the vertical component, $\mathrm{N}_{\mathrm{v}}$, can be obtained by Equation (2)

$$
\begin{gather*}
N_{h}=1.5 K_{\text {dynamic }} * \Delta  \tag{1}\\
N_{v}=N_{h} * \tan \theta \tag{2}
\end{gather*}
$$

Where, $\theta$ is the guy cable angle with horizon. Based on the simplified models of elastic beams on flexible spring supports proposed to develop the condensed flexibility matrix of guyed masts in Ghafari and McClure (2011 \& 2012), simple analytical equations are introduced to calculate shear and bending effects in the mast. Equation (3) is proposed to estimate the maximum bending moment in each span of the mast below the cluster where $\mathrm{N}_{\mathrm{h}}$ and $\mathrm{N}_{\mathrm{v}}$ are calculated ( $M_{\text {max }}$ ), and Equation (4) estimates the maximum shear force in the mast at each cluster level $\left(V_{\max }\right)$, assuming the guy wires are arranged symmetrically in groups of three. The mast stiffness matrix terms and the simple formulas in Equations (1) to (5) would require adjustments if other guying layouts (e.g. four symmetric clusters used with square masts) are used. However, most telecommunication masts use three symmetric clusters in plan view.

$$
\begin{gather*}
M_{\max }=\frac{5}{32} N_{h} L  \tag{3}\\
V_{\max }=\frac{2}{3} N_{h} \tag{4}
\end{gather*}
$$

$L$ is the difference in elevation between the guying level under consideration and the next stay level below it.
The maximum axial compression in the mast can be estimated by superimposing the components due to dead loads and the vertical cable tension components obtained at each cluster level, $N_{v}$. Recognising that the maximum values of these vertical resultant forces are not synchronous, it is recommended to take their root mean square value to provide a less conservative and more realistic estimate. No simplified expression is provided for twisting moments as detailed numerical studies have shown that they are only of secondary importance under earthquake effects (Faridafshin, 2006).

## V. APPLICATION OF THE CONDENSED LINEARIZED METHOD IN ENGINEERING PRACTICE

Guidance on earthquake-resistant design and seismic analysis of communication structures is contained in Appendix M of the Canadian Standard CSA-S37-01(R2013): this is not a mandatory part of the standard yet but the upcoming 2012 edition will make seismic design checks mandatory for post-critical installations that must retain functionality after a design-level earthquake. In the United States, seismic design of telecommunication structures is mandatory in regions where seismic effects may govern over wind effects (ANSI/TIA 222-G). In this context, simplified seismic check procedures for tall masts would prove very useful for tower designers to identify masts with high seismic vulnerability. The proposed method can be implemented as a special module in specialized mast design software used in industry, with due consideration of local seismicity requirements detailed in building codes. Finally, it is to be emphasized that the method is intended for design checks of tall masts in the $150-350 \mathrm{~m}$ range only; taller masts located in moderate to high seismic areas should be analysed with complete nonlinear seismic analysis models. Fig. 8 illustrates the flow chart indicating how the full procedure looks like in the context of a design office and common engineering practice.

Special attention should be paid to the scope and limitations of the method for safe and reliable engineering use. The method is calibrated and validated for tall masts of 150 to 350 m with regular mass and stiffness distribution. Heavy antennas, transmission lines and ancillary components should be considered properly in the lumped mass matrix. Consideration of variability in guy cable pretension forces is of high importance to yield results realistic of field conditions. Frequency analysis is highly recommended to identify mode shapes and frequencies within the sensitive range of common earthquakes.

## VI. conclusions

This technical note confirms the good performance of the linearized analysis method presented by the authors for approximate seismic analysis of tall guyed telecommunication masts. The results presented were obtained from four analysis models of two masts subjected to 81 earthquake records scaled at a peak horizontal ground acceleration value of 0.30 g . The simplified method with condensed symmetric stiffness matrix provides a reliable and conservative estimate of the maximum displacements and cable tension forces of the mast at guy cluster attachment levels. The method is intended for approximate design checks of tall masts in the $150-350 \mathrm{~m}$; taller masts located in moderate to high seismic areas should be analysed with complete nonlinear seismic analysis models.


Fig. 8 Flow chart showing the steps for seismic design checks of guyed telecommunication masts according to the introduced method

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