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Seismic design of lattice telecommunication towers: research, codes and practices

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ABSTRACT

This paper reviews recent research done at McGill University in earthquake-resistant design of lattice towers together with international codes and practices of telecommunication industry. It is largely based on the American Society of Civil Engineers (ASCE) Guide for Dynamic Analysis of Lattice Telecommunication Towers, which should be published in 1999. The Guide is currently being completed by the Task Committee on the Dynamic Response of Lattice Towers of the Technical Committee on Special Structures of the Technical Administrative Committee on Metals of the Structural Engineering Institute. The first author was the main contributor to the Guide for the chapter on seismic input and response.

INTRODUCTION

Many researchers are active in the field of seismic analysis and earthquake-resistant design of buildings. As a result, most building codes and component design standards for buildings contain earthquake-resistant design guidelines that are reasonably detailed. However, latticed telecommunications towers have received far less attention than buildings and designers are left with few guidelines for simplified seismic analysis. In the absence of specific tower documentation, the use of rules proposed by building codes is tempting. However, important towers located in zones of high seismic risk certainly require more scrutiny.

RESEARCH

One of the first reported studies on seismic analysis of self-supporting telecommunication towers was performed by Mikus (1994) based on modal superposition. It was essentially a preliminary study of six relatively short towers, with height ranging from 20 m to 90 m, subjected to three earthquake records. It concluded that the lowest four lateral modes of vibration provided sufficient accuracy in modal superposition analysis.

Gálvez (1995)

A first attempt to propose an equivalent static method for the analysis of lattice self-supporting telecommunication towers was made by Gálvez (1995). The method is also described in Gálvez and McClure (1995): it is based on modal superposition of the lowest three flexural modes of vibration of the tower. The base excitation was a sinusoidal function with maximum amplitude, $\ddot{u}_g$, equal to the peak ground acceleration defined by the National Building Code of Canada (NRC/IRC 1995) for the tower site. This excitation is assumed to be in resonance with each of the lowest three flexural modes of the tower and the modal accelerations are calculated. Using the Square Root of Sum of Squares (SRSS) method, these modal accelerations are combined and the acceleration profile along the tower is estimated. The method makes use of closed-form expressions of natural frequencies and mode shapes developed for prismatic cantilevers, although it is also possible to include more precise tower frequencies and mode shapes.

Detailed analysis using 45 recorded accelerograms (Tso et al. 1992) was used to validate the method for three existing towers of heights of 90 m, 103 m and 121 m. Based on these results, simplified acceleration profiles were proposed depending on the A/V ratio (peak ground acceleration to velocity ratio) of the accelerograms. After trying several shapes of profiles, the simple bilinear profile was deemed the most appropriate. It is defined using truncated modal superposition to calculate the magnitude of the acceleration at the top of the tower ($x = H$) and at elevation $x = 0.8$ H, roughly coinciding with the first inflection point (from the top) in the third transverse mode of the tower. The inertia force distribution is obtained by multiplying the acceleration profile by the mass profile, and the structure is analyzed statically under the effect of these equivalent horizontal forces.

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The comparison between the member forces obtained from detailed dynamic analysis and the static procedure showed an average overestimation of 20% to 30% for the leg members located below x = 0.8 H. However, the results of the static method are not systematically conservative for leg members above that point and for diagonal members in general, and the accuracy is not good with differences in the range of +45% and -70% for horizontal members and -35% to -25% for cross bracings. The method is also limited to the tower geometry used in the study, i.e. a taper ratio (change in width divided by taper height) less than 1:14.5, and a total length to tapered length ratio less than 1.15.

Further work by Zaugg (McClure et al. 1997) has included simulations of eccentric loads due to heavy antenna clusters. It was found that the localized force obtained from the product of the large antenna mass times the amplitude of Gálvez' bilinear acceleration profile provide a good representation of the seismic effects in the antenna mounts and adjacent members. It is suggested that these forces be applied alternatively in various horizontal directions (parallel and normal to the principal directions of the tower), in order to represent the acceleration changes in direction and sign occurring during earthquakes.

Sackmann (1996)

Sackmann (1996) has worked on the prediction of self-supporting lattice tower mode shapes and frequencies, following the work of Gálvez (1995). It was motivated by the fact that further advances in the development of an accurate static method for seismic analysis were hampered by the lack of knowledge of the natural modes and frequencies of lattice towers. His study was based on detailed frequency analysis of ten lattice towers with height ranging from 30 m to 120 m and with equilateral triangle cross-section. All towers were linearly tapered with an equivalent taper ratio, D (Eq. 1), ranging from 0.10 to 0.35, and their top prismatic section did not exceed 20% of the total tower height, H.

$$(1) \quad D = (I_t / I_o)^{1/3}$$

where I_t and I_0 are the second moments of area of the main leg sections at the top and at the base of the tower, respectively. The tower models did not include the mass of antennae or accessories (ladders, transmission lines, ice guards, etc.). The classification presented in Table 1 was used, which is based on three parameters: 1) the a/H ratio of the largest panel (length of truss panel/tower height) - see Fig. 1, 2) the equivalent taper ratio defined in Eq.(1), and 3) the shear coefficient at the base K_{so} (Eq. 2).

$$(2a) \quad K_{so} = 0.29 \frac{a}{H} \sqrt{\Phi}$$

$$(2b) \quad \Phi = 2 \frac{A_L}{A_D} \left(\frac{c}{a} \right)^3 - 1 \quad \textit{for cross braces}$$

$$(2c) \quad \Phi = 8 \frac{A_L}{A_D} \left(\frac{c}{a} \right)^3 \quad \textit{for chevron braces}$$

A_L, A_D, c and a are also defined in Fig. 1. A_L and A_D are the cross-sectional areas of the leg member and diagonal member at the tower base, respectively.

Table 1 Classification of lattice towers according to Sackmann (1996)

	A1	A2	B
a/H	≤ 0.1	≤ 0.1	> 0.1
D	0.1 to 0.2	0.2 to 0.3	0.25 to 0.35
K_{so}	0.1 to 0.25	0.1 to 0.25	0.25 to 0.4

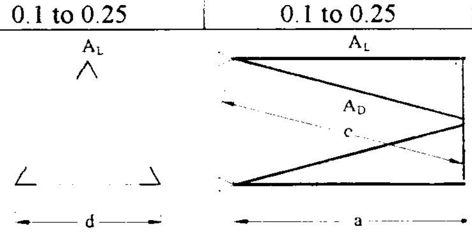

Fig. 1 Definition of symbols in classification of Table 1

Expressions for the fundamental axial mode and the lowest three flexural frequencies and corresponding mode shapes were proposed, as well as estimates of the lowest two torsional frequencies. Details of these expressions are not presented here due to the lack of space. It is noted that Sackmann's tower classification and predictions of the fundamental axial mode and the lowest three flexural modes were later used by Khedr (1998).

Khedr (1998)

The main source of inaccuracy of the static method proposed by Gálvez (1995) lies in the prescribed bilinear shape of the acceleration profile, which may not be a good representation for a wide range of tower geometry. Khedr (1998) has worked on an improved method where the shape of the horizontal acceleration profile is specific to each tower group defined in Table 1. His main contribution was to determine a horizontal acceleration profile that can give the same value of bending moment at each tower section as the value obtained from the SRSS combination of the bending moment resulting from each of the three lowest flexural modes. For the evaluation of the acceleration profile, the self-supporting tower is assumed to be a continuous prismatic cantilever system with rigid base. The mass profile and the lowest three flexural modes of the tower are assumed to be known in a closed form (Sackmann's estimations are appropriate). The expression for the acceleration profile a at any relative position x is given by:

$$(3) \quad a(x) = \frac{a_1 S_{pa1}^2 + a_2 S_{pa2}^2 + a_3 S_{pa3}^2}{\sqrt{a_4 S_{pa1}^2 + a_5 S_{pa2}^2 + a_6 S_{pa3}^2}}$$

where S_{pa1} to S_{pa3} are the spectral acceleration values corresponding to the lowest three flexural modes and the coefficients a_1 to a_6, given in Khedr (1998), vary with the tower classification presented in Table 1.

The verification of Kehdr's method was based on detailed dynamic analysis of ten microwave towers of heights ranging from 30 m to 121 m and subjected to the same set of 45 earthquake records used by Gálvez (1995). Typically, the differences in member forces between the detailed dynamic analysis and the static method are in the range of ± 20%, which is deemed acceptable for approximate design. These differences vary from one tower to another and also from one earthquake to another. The latter is not a problem since in reality a design check would be based on a smoothed design spectrum. For a given earthquake, the variation in the differences obtained lies in the error done in estimating the mode shapes of the towers using the categories defined by Sackmann (1996).

Kehdr (1998) has also proposed a simplified static method for predicting member dynamic forces due to vertical earthquake excitation. Detailed simulations have confirmed that only the first axial mode of vibration is likely to be excited by vertical earthquake accelerations. Therefore, a vertical acceleration profile based on the fundamental axial mode of vibration is sufficient to capture the response of self-supporting telecommunication towers correctly. The acceleration profile along the tower's height can be calculated using the following expression:

$$(4) \quad a_v(x) = S_{av} \times (2.1x + 0.7x^2 - 1.7x^3 + 0.4x^4)$$

where $a_v(x)$ is the absolute acceleration profile in the vertical direction (at position x from the base) and S_{av} is the pseudo-acceleration read from the response (design) spectrum The analysis procedure is similar to the one proposed for the effects of horizontal base motion, except that the force profile obtained from the product of the tower mass and the vertical acceleration profile represents inertia forces in the vertical direction.

Khedr (1998) has also derived simple empirical expressions for earthquake amplification factors, as functions of the tower's fundamental flexural or axial period of vibration, as appropriate. When multiplied by the total tower mass, these factors can be used to estimate the base shear and vertical dynamic reaction. The following two expressions are suggested for evaluating the base shear and vertical reaction:

$$(5) \quad V_h = M \times A_h \times (1.91 - 0.66\, T_f)$$

$$(6) \quad V_v = M \times A_v \times (0.97 + 11.0\, T_a)$$

where V_h is the base shear in N, M is the total mass of the tower in kg, A_h is the peak horizontal ground acceleration in m/s^2, T_f is the fundamental flexural period of vibration in s, V_v is the total vertical reaction in N, A_v is the peak vertical ground acceleration in m/s^2 and T_a is the fundamental axial period of vibration in s. These expressions are valid for towers with regular geometry and heights up to 120 m. They have not been verified for taller towers but it is expected that the trends should be similar. Furthermore, it is important to realize that these expressions are not proposed for detailed member design purposes but only as an approximate tool to assess the seismic sensitivity of towers in the preliminary design phase.

CODES AND PRACTICES

Canadian Standard CSA S37-94

It is only in 1994 that the Canadian Standard CSA S37-94 *Antennas, Towers and Antenna Supporting Structures* has introduced Appendix M that addresses the issue of seismic analysis of lattice telecommunication towers. Since most short self-supporting telecommunication towers are typically of high frequency compared to dominant frequencies of earthquakes, seismic effects are not likely to be significant. The appendix therefore recommends that a frequency analysis of the tower be performed to allow for the identification of the tower's sensitive frequency range. If this range coincides with the frequency content of the dynamic loading, detailed dynamic analysis should be performed. When seismic analysis is required, modal superposition should be used with modal viscous damping ratios between 1% to 3%. The appendix also suggests to use accelerograms or earthquake spectra based on the seismicity levels prescribed by the National Building Code of Canada for the tower site.

Australian Standard AS 3995-1995

The Australian Standard AS 3995-1995 *Design of steel lattice towers and masts* also contains an informative appendix (Appendix C) which includes general guidelines for earthquake-resistant design. In this appendix it is stated that self-supporting lattice towers with height up to 100 m with no significant mass concentrations need not be designed for earthquake effects. However, towers with significant mass and height of more than 100 m or lesser height but with significant mass concentrations, may be exposed to base shears and overturning moments approaching ultimate wind actions. However, the standard does not offer any guidance as to how to estimate the tower response, which suggests that modal superposition analysis is appropriate.

Eurocode 8 Part 3 (ENV 1998-3)

The Eurocode 8 *Design Provisions for Earthquake Resistance of Structures* ENV 1998-3 devotes a complete part to towers, masts and chimneys (Part 3). This part contains a description of basic design requirements, seismic action, modeling considerations and methods of analysis. It is emphasized that the design philosophy of this code is to maintain the function of the structure and to prevent any danger to nearby buildings or facilities. However, the code does not provide protection against damage of non-structural components. The code suggests several methods to describe seismic input: elastic response spectrum, smoothed design spectrum and time-history representations using either artificial accelerograms or recorded strong motions. A complete section is devoted to mathematical modeling, which includes some considerations of the rocking and translation stiffness of the foundation. The code also provides a simplified method of analysis using design spectra in which the base shear is calculated using:

$$(7) \quad F_t = S_d(T)\sum_{i=1}^{n} W_i$$

and this value is distributed along the tower height using the following expression (common in building codes):

$$(8) \quad F_i = \frac{h_i W_i}{\sum_{i=1}^{n} h_i W_i} F_t$$

where W_i is the weight of the i^{th} mass, h_i is the elevation of the i^{th} mass from the base, S_d is the spectral displacement corresponding to the fundamental lateral period of vibration of the tower, T. However the code clearly stipulates that this approach is limited to unimportant structures with height less than 60 m. For other towers, the code recommends the use of modal superposition analysis.

American TS 13 National Earthquake Hazard Reduction Program (NEHRP)

A draft of the American TS 13 National Earthquake Hazard Reduction Program (NEHRP) document for non-building structures (FEMA 1996) suggested a simple design equation for self-supporting telecommunication towers. It recommended that self-supporting telecommunication towers be designed to resist an earthquake lateral force, V, applied at the centroid of the tower and calculated using the following equation:

$$(9)\quad V=\frac{S_{a1}IW}{RT}$$

where V is the lateral force, S_{a1} is the site specific design spectral acceleration at nominal period of 1s, I is the importance factor (I = 1.0 for standard towers and 1.25 for essential or post-critical towers), W is the total tower weight including all attachments, R is a response modification factor and T is the fundamental period of the tower in s. This equation was meant to resemble the maximum base shear equation used in many building codes. However, the basis on which the equation was developed is not clear. It is noted that only the fundamental mode of vibration is considered in this approach, which is not accurate for this type of structure. It was first found in Mikus (1994) and later verified in Gálvez and McClure (1995) and Khedr (1998) that the contributions of both the second and third flexural modes are usually significant.

Uniform Building Code UBC 1997

The Uniform Building Code UBC 1997 devotes a specific section (No. 1632) to earthquake resistant design of non-building structures. However, for telecommunication towers, this section does not specify any procedure different than the one used for building structures. It is suggested that when an approved national standard provides a seismic design procedure for a certain type of non-building structure, such procedure may be used under the following conditions: 1) the seismic zones and occupancy categories must conform with the UBC, and 2) the total base shear and overturning moment calculated must be greater than 80% of the values obtained using the UBC approach. It is noted that the philosophy of the UBC is *to safeguard against major structural failures and loss of life, not to limit damage or maintain function*. This is different than the philosophy behind the provisions for seismic design of towers in which the main concern is to maintain the functionality of the tower.

EIA/TIA Seismic Committee Report (January 1998)

In a recent report released by the seismic committee of the EIA/TIA (Electrical/Telecommunications Industries Association), several recommendations about the seismic analysis of steel antenna towers are made. The objectives of the committee were as follows: 1) to define a methodology for use in seismic analysis of towers, 2) to identify simple but conservative assumptions to make the analysis easier, 3) to provide acceptable methods for more rigorous analysis techniques, and 4) to identify tower characteristics which indicate if seismic analysis should be performed. At present, however, these objectives are not all met in a satisfactory way. The committee recommends to follow the same approach as used in the Uniform Building Code UBC 1997, which is intended for building structures. The total design base shear value is calculated using the following expression:

$$(10)\quad V=\frac{3ZIC}{8}W$$

and

$$(11)\quad C=\frac{1.25S}{T^{2/3}}\le 2.75$$

where V is the total design base shear, Z is the seismic zone factor, I is the importance factor (same as in Eq. 9), W is the total dead load, S is the site coefficient and T is the fundamental period of the tower in s. The calculated base shear is distributed along the tower height in the same manner as prescribed in UBC 1997. The EIA/TIA report also contains a study performed in order to obtain a threshold criterion that can be used to determine whether a seismic analysis is truly needed or not. To this end, a comparison between base shear values obtained from wind effects and seismic effects was done. From this study it was concluded that it is unlikely for seismic effects to control over wind. It should be noted, however, that seismic calculations were carried out using the UBC base shear equation, which makes this conclusion questionable.

CONCLUSION

This paper has summarized recent research efforts at McGill University in order to develop a reliable equivalent static method for the seismic analysis of three-legged self-supporting lattice towers. Standards and codes of practice that address the question are also reviewed, and it is seen that most codes use a building code approach where the fundamental mode of vibration is assumed to dominate the lateral response. A comparison between results of various code approaches and the proposed simplified method (Khedr 1998) will be presented at the Conference for a 72 tons, 121 m high steel lattice microwave tower.

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