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SEISMIC ANALYSIS OF LATTICE TOWERS

Literature Review

by

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1. Introduction

Latticed towers are widely used today as supporting structures, namely to support antennae in telecommunication network systems and in overhead power lines.

Two types of latticed towers are generally used in these applications: guyed towers and self-supporting towers. Self-supporting telecommunication towers are three-legged or four-legged space trussed structures with usual maximum height of 120 m to 150 m. These towers consist of main legs and horizontal and transverse bracings. Main legs are usually composed of 90° angles (in four-legged towers) or 60° schifflerized angles (in three-legged towers). Different bracing patterns are used but the most common ones are the chevron and the cross bracing. Classical steel transmission towers are four-legged latticed structures. The main legs are composed of 90° angles or tubular sections. Transmission towers have several crossarms to support the conductors and ground wires. The bracing patterns used in these towers are similar to those used in self-supporting telecommunication lattice towers.

Self-supporting telecommunication towers are usually designed under the effect of wind and ice loads, without considering earthquakes. These towers may be very important structures in a telecommunication network and the designer should insure that they will perform well in a severe earthquake event, especially for towers located in high risk seismic areas. In the 1994 edition of CAN/CSA-S37 *Antennas, Towers and Antenna-Supporting Structures* (CSA 1994) a new appendix was introduced to address the issue of seismic analysis of self-supporting telecommunication towers. In this appendix it is recommended that, whenever necessary, the tower should be analyzed under the effect of earthquake loading using modal superposition. The base acceleration should be compatible with the values prescribed by the National Building Code of Canada (NBCC 1995) for the tower site. This recommendation is very general, and the designer is left without any specific guidance to assess whether or not a detailed analysis is truly necessary. It would therefore be desirable to rely on a simplified, quasi-static method of analysis to get an estimate of the relative importance of the seismic response of the tower. If the accuracy of such a method can be proven, detailed dynamic analysis may even become unnecessary in the majority of the cases. The objective for the designers is then to compare the effect of earthquake inertia loads to that of extreme wind loads or combined wind and ice loads.

The main design loads in the case of transmission towers are environmental loads including wind and ice or a combination of both. Several extreme cases of loading such as conductor breakage and ice-shedding are usually considered during the design process using equivalent static loads or quasi-static methods. However, earthquake effects are not considered in the design even in high risk seismic areas. There are reports (Pierre, 1995 and Kempner, 1996) of damages in some transmission towers during recent earthquake events. Although in most cases the damage was due to large movements of the tower foundation, it seems relevant to determine the level of stresses these structures are subjected to during earthquakes. The main difference between the behavior of classical transmission towers and self-supporting telecommunication towers arises from the dynamic interaction between the tower and the conductors. If it is possible to simplify this interaction, transmission towers can be treated in a similar fashion as self-supporting telecommunication towers.

2. Dynamic Response of Self-Supporting Lattice Towers

2.1 Response to wind

Chiu and Taoka (1973) studied the dynamic response of self-supporting lattice towers under actual and simulated wind forces. A three-legged, 46 m tall self-supported lattice tower was instrumented to study its dynamic response to wind forces. The tower was then idealized as a space truss with masses lumped at the horizontal panel points. The analysis of the field measurements indicated that the measured dynamic properties of the tower agreed with the calculated values. It was also found that the assumption of uncoupled motion in the two principal horizontal directions is valid. The study showed that for free-standing structures the fundamental mode of vibration is predominant. The average damping for the fundamental period was found to be 0.5% of the critical viscous damping value.

Venkateswarlu et al. (1994) studied the response of microwave lattice towers to random wind loads. The dynamic response was predicted using a stochastic approach. A spectral approach was proposed for calculating the along-wind response and the gust response factor. The gust response factor is defined as the ratio of the maximum expected wind load effect in a specified time period to the corresponding mean value in the same time period. A free-standing four-legged tower of 101 m height was used in this study. The variation of the gust response factor along the height of the tower was calculated with and without the contribution of the second and higher modes. It was found that the maximum contribution of the higher modes of vibration to the gust response factor is only of 2%. The gust response factor obtained using the proposed stochastic method varied between 1.547 and 1.584 along the height. Using the formulae recommended by the Indian (IS:875-1987), Australian (AS 1170-2-1989), British (BS 8100-1986) and American (ASCE 7-88-1990) standards, the values of the gust response factor were found to be 2.03, 2.21, 1.93 and 1.89 respectively. Comparing these results, it was concluded that the standards values were 20% to 40% higher than the values obtained using the spectral method.

In a series of papers on the along-wind response of lattice towers, Holmes (1994,1996) evaluated expressions for the gust response factor in a closed form for both the shearing force and the bending moment along the height of the tower. The tower used in this study was idealized with linear taper and a uniform solidity ratio so that the drag coefficient was kept constant. The mass per unit height of the tower was assumed to vary according to the following relation:

$$m(z) = m_o(1 - k(\frac{z}{h})^{\gamma}) \qquad (1)$$

where,

h = total height of the tower

m_o= mass of the tower at the base

k and γ are constants determined so that $m(z)$ best fits the actual mass distribution

In this study, only the effect of the lowest flexural mode was considered, which was assumed to be in the form:

$$\mu_1 = (\frac{z}{h})^{\beta} \qquad (2)$$

where β is a constant determined so that μ_1 best fits the calculated lowest flexural mode. Using the previous assumptions, the expressions for the gust response factors for both the shearing force and bending moment were obtained in a closed form as follows:

$$G_q = 1 + \frac{r[g_B^2 B_s F_2 + g_R^2 (SE/\xi) F_3 F_4 F_5]^{\frac{1}{2}}}{F_1} \tag{3}$$

and

$$G_m = 1 + \frac{r[g_B^2 B_s F_7 + g_R^2 (SE/\xi) F_3 F_4 F_8]^{\frac{1}{2}}}{F_6} \tag{4}$$

where G_q and G_m are the gust response factors for the shear and the bending moment along the height of the tower respectively. The reader may refer to Holmes (1994,1995) for a complete explanation of the terms used.

These expressions were then compared to the expressions currently used and both were found in agreement. The advantage of the proposed over the currently used expressions is the inclusion of more factors to account for the effects of various parameters associated with both the wind and the structure. An expression for the aerodynamic damping of the tower, due to the relative motion between the tower and the wind, as a ratio of the critical damping was also derived. In addition, a closed-form expression for the deflection at the top of the tower was derived considering three components of deflections namely the mean, background and resonant components. A study was then performed to investigate the effects of the height, taper ratio, and mean velocity on the gust response factors for shear force and bending moments. Finally the work was extended to predict an effective static load distribution, including the mean, background fluctuating and resonant components of the wind.

2.2 Seismic response

One of the first studies discussing earthquake effects on antenna lattice towers was presented by Konno and Kimura (1973). The study aimed at collecting information on tower mode shapes, natural frequencies and damping properties. One of the towers used in this study was instrumented when the 1968 Off-Tokachi earthquake occurred and the data collected was analyzed and compared with simulated results. The numerical simulation used a stick model of the tower with lumped masses and a viscous damping ratio of 1%. In some members the forces due to earthquake loading were found to exceed those due to wind. In fact, some local damage and permanent deformations were observed at the base of the tower after the earthquake.

More recently, Mikus (1994) studied the seismic response of self-supporting telecommunication towers. The aim of this study was to improve the understanding of the response of these towers to earthquakes. Six towers with height from 20 m to 90 m were used in the study: bare towers only, no antennas, attachments, ancillary components etc. Three earthquake records were selected as the base excitation. A detailed dynamic analysis was performed using modal superposition, and it was concluded that the use of the lowest four modes of vibration provided sufficient accuracy. The frequency of the first axial mode of the towers was found to be in the range of 11 to 43 Hz. This range was either not included in the frequency content of the earthquake records used or corresponded to small amplitudes of input accelerations. It was therefore concluded that the vertical component of the earthquakes has negligible effect on the towers studied.

A first attempt to find an equivalent static method for the analysis of latticed self-supporting telecommunication towers was made by Gálvez (1995). The method was based on modal superposition taking the effect of the lowest three flexural modes of vibration into consideration. As self-supporting towers behave essentially as cantilever beams, Gálvez suggested the use of natural frequencies and mode shapes expressions developed for prismatic cantilevers. The effects of taper ratio and shear deformations were included by means of correction factors to the classical solution for prismatic Euler cantilevers. The expression for the natural frequencies was given as:

$$f_i = \frac{\lambda_i^2}{2\pi L^2}\sqrt{\frac{EI_o}{m_o}} \tag{5}$$

where,

EI_o= flexural rigidity at the tower base

m_o= mass per unit length at the tower base

L_o= tower height

The parameter λ_i (essentially a dimensionless frequency) was given as:

$$\lambda_i = \lambda_b F_{ct} F_{cs} \tag{6}$$

where,

λ_b= frequency parameter for prismatic cantilever

F_{ct}= taper correction factor

F_{cs}= shear correction factor

The expression for the flexural modes of vibration was given as:

$$\phi_i(x) = \cosh(\lambda_i x/L) - \cos(\lambda_i x/L) - \sigma_i[\sinh(\lambda_i x/L) - \sin(\lambda_i x/L)] \tag{7}$$

where,

$$\sigma_i = \frac{\sinh\lambda_i - \sin\lambda_i}{\cosh\lambda_i + \cos\lambda_i} \tag{8}$$

The base excitation was assumed to be a sinusoidal wave with the maximum amplitude, $\ddot{u}_g$, being equal to the peak ground acceleration defined by the National Building Code of

Canada (NBCC 1995) for the tower site. The acceleration response for a harmonic ground motion was defined by:

$$\ddot{u}(x,t) = \ddot{u}_g \Sigma_{i=1}^{3} \phi_i(x)\psi(\lambda_i)\ddot{a}(\Omega,\omega_i,t) \tag{9}$$

where, Ω = frequency of the exciting force

$$\psi(\lambda_i) = 2\sigma_i/\lambda_i \tag{10}$$

$$\omega_i = 2\pi f_i \tag{11}$$

For $\Omega \neq \omega_i$ and using the frequency ratio $\beta_i = \Omega/\omega_i$,

$$\ddot{a}(\Omega,\omega_i,t) = \frac{\beta_i}{1-\beta_i^2}(\sin\omega_i t - \beta_i \sin\Omega t) \tag{12}$$

At resonance in mode i, $\Omega = \omega_i$,

$$\ddot{a}(\Omega,\omega_i,t) = \tfrac{1}{2}(\sin\omega_i t + \omega_i t\cos\omega_i t) \tag{13}$$

The input excitation was assumed to be in resonance with each of the lowest flexural modes considered. Using the SRSS method for combining the relative modal accelerations, an acceleration profile along the height of the towers was defined. Detailed dynamic analysis using a total of 45 base accelerograms was used to validate the method using three different towers. Based on these results, simplified acceleration profiles were proposed depending on the A/V ratio (peak ground acceleration to velocity ratio) of the accelerograms. The inertia force was simply found by multiplying the acceleration profile by the mass profile. The structure was then analyzed under the effect of these equivalent "static" inertia forces. Although simple, the method did not always give good estimates for the internal forces. For the main legs in general, the method gave conservative values accurate enough for preliminary design. Results were not systematically accurate and conservative for other members. The margin of error for the force prediction in the horizontal bracing was between -68% and +43%, and for cross bracing it was in the range of -35% to +23%. The method was limited to the tower geometry used in the study, i.e. having a taper ratio less than 14.5, and a total length to tapered length ratio less than 1.15.

A draft of the TS 13 Nonbuilding structures document for the United States National Earthquake Hazard Reduction Program (US NEHRP) was released for comments in 1996. In this draft a simple design basis for self-supporting telecommunication towers subjected to earthquake loads was suggested. It was recommended that self-supporting telecommunication towers be designed to resist an earthquake lateral force applied at the centroid of the tower and calculated using the following equation:

$$V = \frac{S_{a1} I W}{R T} \tag{14}$$

where,

V = lateral force

S_{a1} = site specific design spectral acceleration at period of 1 second

I = importance factor (I=1.0 standard towers, 1.25 essential towers)

W = total dead weight including all attachments

R = response modification factor

T = fundamental period of the structure in seconds.

This equation was meant to be compatible with the base shear equation used in most building codes. However, the basis on which the equation was developed is not clear. The choice of the value of S_{a1} corresponding to 1 second is not justified. Also only the fundamental mode of vibration is considered which is not accurate for this type of structure in which the contribution of both the second and third flexural modes are usually significant (Mikus, 1994).

3. Dynamic Response of Transmission Line Structures

The study of the dynamic problem arising from the unique case of the tower-conductor coupled system attracted several researchers. Some researchers investigated the dynamic loads on transmission towers due to galloping of the conductors (Baenziger et al. 1994), conductor breakage (McClure and Tinawi 1987), ice shedding from the cables (Jamaleddine et al. 1993) and the free vibration of the coupled system (Ozono et al. 1988). However, most of the work done for the seismic analysis of the transmission lines involved the tower alone without considering the coupled tower-conductor problem.

A review of the dynamic problem of the coupled tower-conductor system is indicated before summarizing the work done in seismic analysis of the transmission line systems. Ozono and Maeda (1993) studied the in-plane dynamic interaction between the tower and the conductors. The tower-conductor system was simplified assuming the tower to be a single lumped mass cantilever. Two models were used, the first with two spans of conductors and the second with only span. For the two models, the conductors' ends not attached to the suspension tower were fixed to a rigid wall. The contribution of the natural modes of the conductors to the tension force exerted on the tower was obtained and it was found that at lower frequencies (less than 2 Hz) the contribution of the transverse wave modes to the tension force exerted on the tower is dominant. The results of this investigation suggested that the conductors play two important roles in the coupled system. The first effect is that when the conductors vibrate locally at a dominant frequency, their deformation induces a dynamic tension force on the tower. The second effect is that when the tower vibrates at a dominant frequency the conductors act as massless linking springs in the coupled system, i.e. their inertia effects are not significant.

In an earlier study, Long (1974) investigated the effect of seismic excitation on a transmission tower neglecting the effects of the overhead conductors. The study was extended to evaluate the forces exerted by the conductors on the tower. The steel transmission tower was divided into two parts. The top part consisted of the prismatic part and the cross arms, and was treated as a flexible with uniform stiffness and mass, and treated as a uniform cantilever. The bottom part was assumed to be a rigid lumped mass. The total displacement of the flexible cantilever was then given by the following equation:

$$u(x,t) = z(t) + \Sigma_{k=1}^{\infty} h_k(x) f_k(t) - \frac{ml^4}{EI}\ddot{z}(t)[c_1(x) - \Sigma_{k=1}^{\infty} \frac{h_k(x)}{\lambda_k^4}] \tag{15}$$

where,

EI= flexural rigidity

m= mass per unit length

$z(t)$= ground displacement

$h_k(x)$= deflection curve for normal mode of vibration k

$f_k(t)$= displacement response to the ground motion of a simple oscillatory system in mode k

l= length of cantilever

$c_1(x)$= deflection due to static uniform loading $= \frac{1}{24}(\frac{x}{l})^4 - \frac{1}{6}(\frac{x}{l})^3 + \frac{1}{4}(\frac{x}{l})^2$

λ_k= dimensionless frequency, positive root of the equation $1 + \cosh\lambda \cos\lambda = 0$ for mode k

k= mode number

The three terms of equation (15) are the horizontal ground displacement, the displacement response of the structure to the ground motion, and a deflection resulting from the difference of acceleration loadings of ground motion and free vibration. The deflection at the top of the tower was evaluated using equation (15), assuming that the maximum values of each of the three terms in the equation occurred simultaneously. The response spectrum technique was used to evaluate the maximum value of the response function $f(t)$, and the maximum value of the ground displacement $z(t)$ was obtained from the earthquake records. It was concluded that the entire tower moved rigidly with the ground and that no amplification of stresses was produced by the ground motion. The second part of the study aimed at calculating the force exerted by the conductors on the tower due to the earthquake excitation. Three directions of earthquakes were considered namely, transverse, longitudinal and vertical. The forces calculated in the three cases were found to be very small and could be resisted safely. It should be noted that the tower used in this study was a very rigid one having a lowest frequency of vibration of 5 Hz.

Kotsubo et al. (1985) performed dynamic tests on three transmission towers before and after installation of the conductors. The purpose of the study was to determine the effects of the conductors on the dynamic characteristics of the towers. The earthquake response of the towers was then evaluated numerically. The three towers used were two strain towers (with conductors directly anchored to the tower) with heights of 92.5m and 68.5m, and a suspension tower with height of 92.2m. The results were reported for the case of the suspension tower only. The modes of vibration of the tower were calculated using both a plane truss model and a space truss model. Ambient vibration measurements for the tower were carried out before the installation of the cables. From these measurements and using power spectra, the natural frequencies, modes of vibration and damping properties were obtained. After the installation of the cables, vibration tests using an exciter were carried out. The exciter was set up on the third arm from the top of the tower. It was observed that there were no significant changes in the natural frequencies and the modes of vibration of the tower before and after the cable stringing, which suggested that the dynamic interaction between the cables and towers is insignificant for suspension towers. The damping ratio of the tower was found to be in the range of 0.2 to 2.0% of the critical viscous damping. The earthquake responses were then calculated using the plane truss model and the space truss model ignoring the presence of the cables. For the plane truss model, the responses were calculated for both the longitudinal and the transverse direction to the transmission line. It was concluded that it is sufficient to model the tower as a plane truss.

In a more recent study conducted by Li et al. (1991) mechanical models for long-span transmission line systems under earthquake effects were presented. This study included the derivation of mass and stiffness matrices for the tower-cable coupled system for the longitudinal and transverse directions. For the vertical direction the mass of the conductors was calculated and then lumped at the appropriate joints. For each of the three main directions a dynamic analysis was carried out using three earthquake records namely Qian'an (China), ElCentro (USA) and Ninghe (China). The analyses were done for the following three cases for comparison:

I- The discretized model of the tower without the conductors.

II- The discretized model of the tower with the mass of the conductors lumped at relevant tower joints.

III- The coupled tower-conductor model.

It was found that for the vertical ground motion the seismic response of model II is greater than that of model I. For both the lateral and longitudinal ground motions, the response of model III was greater than that of model II, which in turn was greater than that of model I. It was concluded that the effects of the conductors on the seismic response of their supporting tower are not negligible and should be taken into consideration.

As a very crude approximation, Kempner (1996) suggested analyzing the tower statically under the effect of lateral force acting at the tower's center of mass, using the same equation presented in the case of self-supporting telecommunication tower.

$$V = \frac{S_{a1} I W}{R T} \tag{14}$$

The definitions of the terms used in the previous equation are the same as for the case of self-supporting towers, except that W is the total dead load without including the weight of the supported wires. The lateral force is applied in both the longitudinal and transverse directions. If it is found that earthquake loading is likely to govern the design a more detailed lateral force distribution or modal analysis, as specified by IEEE 693 "Seismic Design of Substation Structures", are suggested.

4. Seismic Response of Tower-Shaped Structures

Due to the little amount of literature available on seismic analysis of latticed self-supporting towers, the search was directed towards other structures that behave essentially as cantilevers, namely offshore towers and intake-outlet towers. The aim of this search is to gain insight of the approaches used in analyzing such structures under seismic excitations and to find if a simplified method for analysis is available.

4.1 Seismic response of offshore towers

Penzien and Kaul (1972) studied the response of offshore towers to strong motion earthquakes. In their work, the response spectrum method of analysis was used and compared with the proposed stochastic method. In this proposed method, a mean ergodic Gaussian process of finite duration was used as the stochastic model for the horizontal ground acceleration. The aim of the study was to determine the transverse shear distribution and the overturning moment along the height of the towers without investigating the individual member forces. The towers were modeled as stick models with seven joints along the height on which the mass of the tower was lumped. A condensed stiffness matrix corresponding to the lateral displacements of the model was evaluated, and from the mass and stiffness matrices of the model, the eigen properties of the towers (frequencies and mode shapes) were predicted. The distributions of the transverse shear and overturning moment were then calculated using the response spectrum of the earthquake excitation considering the contribution of the lowest three flexural modes. The results were found to be comparable to those obtained with the more rigorous stochastic approach.

Anagonstopoulos (1982), in his work on modal solutions for the earthquake response of offshore towers, concluded that modal superposition gives good estimates of the overall response of the towers. For some members, however, the estimated value of the bending moment was in an error of about -60%, yet the difference in total stresses were less than 13% which can be reduced by increasing the number of modes in the summation. Due to the uncertainties in the earthquake loading, Anagonstopoulos suggested the use of more earthquake excitations instead of increasing the number of modes in the analysis. He also suggested that the inclusion of the lowest three modes in each of the three principal structural directions (the two horizontals and the vertical) would be adequate for design purposes.

In the work reported by Chan (1987), response spectrum techniques for multi-component seismic analysis of offshore platforms were evaluated. Two platforms were modeled taking into account the added mass of water. In this study three components of earthquake input were considered, two horizontal components with the ratio 0.67 : 1.0 and a vertical component with 0.5. The study aimed at evaluating the techniques used for modal combination as well as seismic component combination rules. The member forces and stresses calculated using different combination rules for both the modal summation and seismic components were compared with those obtained using detailed direct integration analysis. The different modal combination rules studied were the Square Root of Sum of Squares (SRSS), the Complete Quadratic Combination (CQC), and the American Petroleum Institute (API) method. For different directional seismic inputs, the SRSS and the Multi Component Quadratic Combination (MCQC) rules were used. It was concluded that all of these combination rules gave comparable results, and the CQC-SRSS rule was recommended because of its conservative results. As part of his study, Chan also checked the error resulting from neglecting the effect of higher modes (above the eleventh mode) in the analysis. He concluded that because all lower modes are horizontal, the vertical forces could be underestimated by a truncated analysis which in turn would affect the support design.

4.2 Seismic response of intake outlet towers

Valliappan et al. (1980) investigated the effect of earthquakes on the intake tower of Magrove Creek dam in Australia, using both dynamic and pseudo-static analyses. The design spectrum approach was used as a basis of the pseudo-static analysis considering only the lowest flexural mode of vibration. The mode shape used was that reported in Clough and Penzien (1993) in the form of a cosine function. The structure was then analyzed statically under the effect of inertia forces resulting from multiplying the acceleration profile due to the first mode shape by the mass. Detailed dynamic analysis was then performed and the results obtained for both analyses were compared. From this comparison, it was concluded that the pseudo-static analysis considering the lowest flexural mode is only an approximate solution. However, this conclusion might change if higher modes were included.

A simplified method for seismic analysis of intake-outlet towers was developed by Chopra and Goyal (1991). The method was used to estimate the maximum forces in these towers using the design earthquake spectrum. A simplified step-by-step procedure based on the Stodola and Rayleigh methods for the calculation of the lowest two natural periods was suggested. It was demonstrated that considering the lowest two flexural modes of vibration is accurate enough for the preliminary design phase. The procedure can be summarized in the following steps:

1. Definition of a smooth design spectrum suitable for the site of the tower.

2. Calculation of the added mass associated with both the inside and outside water.

3. Definition of the structural properties of the tower:

   a. Mass per unit height $m_s(z)$

   b. Flexural rigidity, $E_sI(z)$ and shear rigidity, $G_sK(z)A(z)$

   c. Modal damping ratio, ξ_n

4. Calculation of the lowest two natural periods of the tower using the proposed simplified step-by-step procedure.

5. For each mode of vibration, the lateral force distribution was predicted using a generalized single degree of freedom approach as follows:

   a. Determine the pseudo acceleration ordinate S_a from the design spectrum corresponding to period T_n and damping ratio ξ_n .

   b. Calculate the generalized mass M_n and the generalized excitation term L_n using the following expressions:

$$M_n = \int_0^{H_s} m_s(z)[\phi_n(z)]^2 dz \tag{16}$$

$$L_n = \int_0^{H_s} m_s(z)\phi_n(z) dz \tag{17}$$

where, H_s = tower height

$\phi_n(z)$ = lateral displacements of the tower in the n^{th} vibration mode

   c. Calculate equivalent lateral forces $f_n(z)$ using the following expression:

$$f_n(z) = \frac{M_n}{L_n} S_a(T_n, \xi_n) m_s(z)\phi_n(z) \tag{18}$$

6. The maximum shear and bending moment at any section along the tower height were then found using the SRSS modal combination method.

It is noted that the step-by-step method for estimating the lowest two natural periods is accurate if the variation in the tower cross-sectional properties can be expressed in a closed form. Since self-supporting lattice towers have usually irregular changes in their cross-sectional properties, the use of this method will only give very crude estimates for the natural periods. Also, a computer program was suggested for the implementation of the proposed procedure , which means that it is not such a "simplified" procedure.

5. Design Code Approaches for Seismic Analysis

Different design code approaches for the analysis of structures under earthquake loads need to be reviewed. Two types of structures are considered here, namely safety-related nuclear structures and buildings.

The seismic analysis of safety-related nuclear structures standard of ASCE (1986) suggests acceptable methods for the analysis and provides the methodology and the input ground motion to be used in calculating the response of such structures. This standard defines two methods for specifying the seismic input, namely response spectrum and input ground motion time history. The horizontal component of the response spectral ordinates (absolute acceleration S_a, spectral velocity S_v, and spectral displacement S_d) are obtained by applying dynamic amplification factors to the corresponding maximum values of ground motion (acceleration a, velocity v, and displacement d) obtained from the response spectrum. These amplification factors depend on the amount of damping and are given as ratios of S_a/a, S_v/v, and S_d/d. The standard requires the use of two equal horizontal earthquake components. Two thirds of the horizontal component value is used as the vertical component of the input. If time histories are used, three different earthquake records should be used in three orthogonal directions. These records must be selected so as to represent the site conditions.

The standard recognizes four methods for the analysis of such structures: the direct integration method, the response spectrum method, the complex frequency method and the equivalent static method. The first three methods are well documented in textbooks (Bathe 1982, Gupta 1992, and Clough and Penzien 1993) and need not be reviewed here. As for the equivalent static method, the standard restricts its use to cantilever models with uniform mass distribution. Multi-degree of freedom models (MDOF) of cantilevers with non uniform mass distribution can be analyzed using the static method if the maximum response is expected to result from loads in the same direction. In this case, the equivalent static load is determined by multiplying the structure's mass profile by a constant acceleration equal to 1.5 times the peak acceleration of the response spectrum. For cantilever structures with uniform mass, values of 1.0 and 1.1 applied to the peak spectral acceleration are used to determine the tower base shear and base moment respectively. The justification of these values is not presented in the standard. The total response for the three components of seismic input is then obtained using the SRSS combination rule.

Although the standard recommends this procedure for MDOF models, the equivalent static method is limited to very simple models which have a dominant lowest frequency mode of vibration.

The usual approach suggested in building codes (Paz 1994) for seismic analysis is to evaluate a global base shear value. The base shear is then distributed along the height of the structure assuming that the lowest mode of vibration is dominant and that the lateral displacement varies linearly. The National Building Code of Canada (NBCC 1995) specifies the minimum base shear for which the structure should be designed, by the following equation:

$$V = \frac{V_e}{R} U \tag{19}$$

where,

R = force modification factor

U = calibration factor = 0.6

V_e= equivalent lateral seismic force

The value of V_e is given by the following relation:

$$V_e = vSIFW \tag{20}$$

where,

v = zonal velocity ratio derived from the probabilistic study of the ground motion

S = seismic response factor which is a function of the fundamental period of the structure and the relative values of the velocity zone Z_v and the acceleration zone Z_a

I = importance factor

F = foundation factor, which depends on the soil conditions at the site

This base shear force is in equilibrium with distributed seismic forces along the height of the structure, given by:

$$F_x = V\left(\frac{W_x h_x}{\sum_{i=1}^{n} W_i h_i}\right) \tag{21}$$

where W and h are the floor weight and height, respectively and n is the number of stories.

It should be noted that the code recommends dynamic analysis using the response spectrum method and modal techniques for buildings with irregular shapes.

6. Conclusions

From this literature review it can be seen that seismic analysis of self-supporting telecommunication towers has received very little attention. The work done in other fields cannot directly be applied to self-supporting latticed towers. Since the designers are left without much guidance to assess if a detailed dynamic analysis is required, earthquake effects are usually ignored in the design office. For short towers and low risk seismic area this may be acceptable. However, in high risk seismic areas and for tall towers the designer should be able to perform at least a simple quasi-static analysis as a quick design check. Therefore, a simplified quasi-static method is proposed. The method is based on the modal superposition method and the response spectrum approach. It is anticipated that the proposed method will give reliable estimates of the member forces and in most cases performing a detailed dynamic analysis will become unnecessary.

Analyses of transmission towers under seismic excitation were conducted for particular systems such as very rigid towers, suspension towers with relatively light conductors and very long spans with heavy conductors. The researchers were divided among themselves into two extremes. The first one neglected the tower-conductor interaction effects, and even went further by not including the mass of the conductors. The second extreme was the recommendation of a detailed analysis of the tower-conductor system. As bare transmission towers are almost equivalent to self-supporting telecommunication towers, part of this work will be devoted to trying to find an equivalent added mass to replace the effect of the conductors. If this is feasible, the work will be extended to investigate the applicability of the proposed simplified method for telecommunication towers to transmission towers.

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