Structural Engineering and Mechanics, *Vol. 69, No. 5 (2019) 567-577*
DOI: https://doi.org/10.12989/sem.2019.69.5.567

Static and dynamic analysis of guyed steel lattice towers

Hussam M. Meshmesha[1a], John B. Kennedy[2b], Khaled Sennah[3c] and Saber Moradi[*3]

[1]*Formerly: Bantrel Co., Calgary, Alberta, Canada*
[2]*Department of Civil & Environmental Engineering, University of Windsor, Windsor, Canada*
[3]*Department of Civil Engineering, Ryerson University, Toronto, Ontario, Canada*

(Received March 16, 2018, Revised January 24, 2019, Accepted January 28, 2019)

Abstract. Guyed steel lattice towers (or guyed masts) are widely used for supporting antennas for telecommunications and broadcasting. This paper presents a numerical study on the static and dynamic response of guyed towers. Three-dimensional nonlinear finite-element models are used to simulate the response. Through performing static pushover analyses and free-vibration (modal) analyses, the effect of different bracing configurations is investigated. In addition, seismic analyses are performed on towers of different heights to study the influence of earthquake excitation time-lag (or the earthquake travel distance between tower anchors) and antenna weight on the seismic response of guyed towers. The results show that the inclusion of time lag in the seismic analysis of guyed towers can influence shear and moment distribution along the height of the mast. Moreover, it is found that the lateral response is insensitive to bracing configurations. The results also show that, depending on the mast height, an increased antenna weight can reduce the tower maximum base shear while other response quantities, such as cables tension force are found to be insensitive to variation in the antenna weight.

Keywords: bracing configurations; dynamic analysis; guyed lattice towers; seismic analysis; ultimate load; finite-element

1. Introduction

The need for guyed telecommunication towers has accelerated with the requirements for effective communications in particular with the exponential growth in the use of wireless communication tools. Past earthquake records have shown that telecommunication facilities can be damaged in earthquakes (FEMA 1991). The Canadian standard for Antenna Towers and Antenna-Supporting Structures, CSA-S37-13 (2013) provides guidance on seismic analysis of lattice towers. It introduces mandatory seismic checks for all designated post-critical structures that must remain serviceable immediately after an earthquake. Other international codes and standards (such as ANSI/TIA-222, ASCE/SEI 10-15, and BS 8100) have also recognized the importance of considering dynamic effects in the design of antenna supporting structures.

The dynamic analysis of antenna supporting towers have been explored in previous research (for example, De Macedo 2016, Ismail 2016, Mahboba *et al.* 2013, Ghafari and McClure 2012, Desai and Punde 2011, Faridafshin and McClure 2008, De Oliveira *et al.* 2007, Hensley and Plaut 2007, Hensley 2005, Amiri 2002). Among others, Shi and Salim (2015) developed nonlinear finite element models for the static and dynamic analysis of guyed towers. Wu *et al*. (2014) reported that near-fault pulse-like ground motions impose a larger seismic response to freestanding or self-supporting tower systems compared to far-field ground motions. Yang *et al.* (2016) used incremental dynamic analysis and nonlinear static pushover analysis to estimate the response of tower-line systems under downburst wind loading. Meshmesha *et al.* (2006) proposed using a simple beam-column element for static, free, and forced vibration analyses of lattice structures. Lacarbonara and Ballerini (2009) proposed a passive damper for vibration mitigation of guyed masts. Law *et al.* (2006) studied the dynamic response of a 50 m guyed mast under the action of Typhoon Dujuan. A new method was proposed by Oskoei and McClure (2011) for evaluating the equivalent dynamic stiffness of guy clusters for simplified seismic analysis of tall guyed telecommunication masts. Several researchers have evaluated the response of latticed steel towers under wind loading. Among others, Battista *et al.* (2018) studied the wind response and fatigue life of tall and slender telecom steel towers with double controllers. Khan *et al.* (2004) performed reliability analysis of latticed towers. Sparling and Wegner (2007) investigated approximate methods for estimating peak wind-induced load effects in guyed telecommunication masts. The response of inclined cables has been also investigated in the literature. For example, Li *et al.* (2013) presented three-dimensional non-linear dynamic equations for inclined cables and analyzed the vibration modal of three-span suspended cable structures. Xia and Cai (2011) presented an equivalent stiffness method to analyze the sag effect for stay cables. Wu *et al.* (2007) derived equations of motions for an

∗ Corresponding author, Assistant Professor
E-mail: s.moradi@ryerson.ca
[a]Senior Engineer (Passed away)
[b]Emeritus Professor
[c]Professor

http://www.techno-press.com/journals/sem&subpage=7
ISSN: 1225-4568 (Print), 1598-6217 (Online)

inclined cable to study the effect of cable loosening on the nonlinear parametric vibration of inclined cables.

Based on reports from past earthquakes, the number of damaged transmission towers has not been large, which has led to a general perception that steel lattice towers are not vulnerable to earthquakes and their design is governed by other environmental effects, such as wind loading (EPRI 2009). However, it is argued that the available reports from past earthquakes are not reflecting the actual extent of seismic damage to transmission towers (Madugula *et al.* 2001, De Macedo 2016). Moreover, these reports are mostly on the performance of self-supporting transmission towers not guyed lattice masts (guyed lattice towers) that rely on guy cables for their lateral stability. The seismic response of guyed masts, which are different from self-supported towers, has not been thoroughly investigated (De Macedo 2016).

Therefore, more studies are required to investigate the effect of potentially important parameters on the behavior of these towers under static and dynamic loads. The first part of this paper aims at studying the effect of using different bracing configurations on the static and dynamic response of guyed towers. This is considered as one of the practical issues that may affect the cost and the weight of guyed tower structures. The second part of the paper deals with the delay time of applied seismic excitation; it examines the effect of earthquake travel distance on the seismic response of guyed towers of different heights. When different structural supports are excited by different or delayed signals, this leads to complex dynamics, and can have particular significance on structures with large distances between their supports. Finally, the effect of a lumped mass at the top of the mast on the seismic response of guyed towers is studied.

2. Static and modal analyses: Effect of bracing configuration

The research by Pratt (1988) was the first attempt to study the effect of bracing configurations on the performance of latticed mast structures on 11 two-dimensional latticed bracing configurations, shown in Fig. 1. The structures were subjected to lateral static loads at the top of the mast. They concluded that the diamond and the St. Andrew’s cross bracing configurations, shown in Fig. 1(a) and (c), exhibited the highest strengths. The research by Ellis *et al.* (1998) extended Pratt’s study to analyze free-standing latticed masts for Canadian warships. The aim of their study was to reduce the weight and profile area of these structures, evaluating two bracing configurations, shown in Fig. 2. The Marshall bracing configuration, Fig. 2(b), was based on the diamond bracing configuration with off-setting. The St. Andrew’s cross, Fig. 2(a), was used as the control bracing configuration. The masts were self-standing and with squared cross sections. The above-mentioned studies dealt with the self-supporting lattice towers. The effect of different bracing configurations on the behavior of tall-guyed towers has not been investigated. In the present study, the effect of using five different bracing configurations, shown in Fig. 3(a), on the response of a 364.5 m tall-guyed tower is investigated using modal analysis and horizontal static loading. This tower height was used as a representative for tall-guyed towers since the height of such tall towers ranges from 200 m to 600 m. This study includes the Marshal bracing configuration (CSA-S37-01) with a ratio of an off-setting length to the tower’s panel width of 0.02.

Table 1 Material properties of the 364.5 m guyed tower

	Property	Unit
Area of legs	0.022	m^2
Area of diagonals	0.008	m^2
Area of horizontals	0.0014	m^2
Area of guy cables	0.0027	m^2
Modulus of elasticity	200	GPa
Mast mass per unit length	594	kg/m
Average mass per unit length for cables	21	kg/m
Total weight of guyed mast	3,188	kN

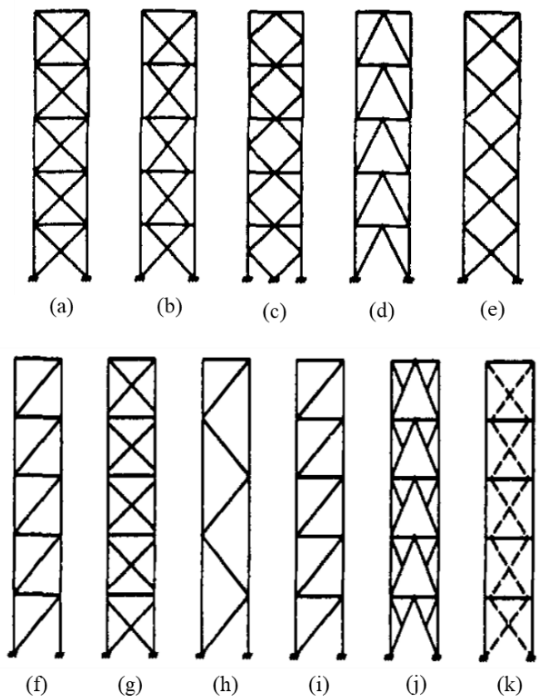
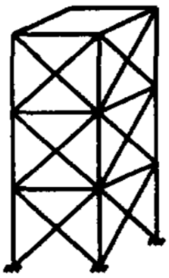


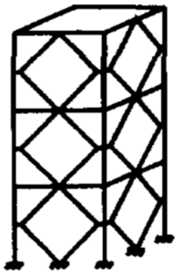
Fig. 1 Various two-dimensional latticed bracing configurations in Pratt’s study (1988)

2.1 Tower geometry used in the bracing configuration study

A 364.5 m tall galvanized steel guyed tower of triangular cross section (i.e., with three legs) pinned at their base was considered in this study. All members were of solid round steel section. The mast was laterally supported by pre-tensioned guys, connected to the legs of the mast and anchored to the ground at equal angles. Five different bracing configurations, shown in Fig. 3, were studied. The profile of the guyed tower is shown in Fig. 4 while Table 1

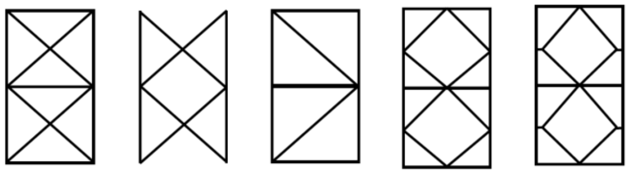


(a)



(b)

Fig. 2(a) St. Andrew’s cross and (b) Marshall’s configuration



(a) (b) (c) (d) (e)

Fig. 3 Various bracing configurations considered in the present study for (a) St. Andrew’s, (b) X, (c) K, (d) Diamond, and (e) Off-setting

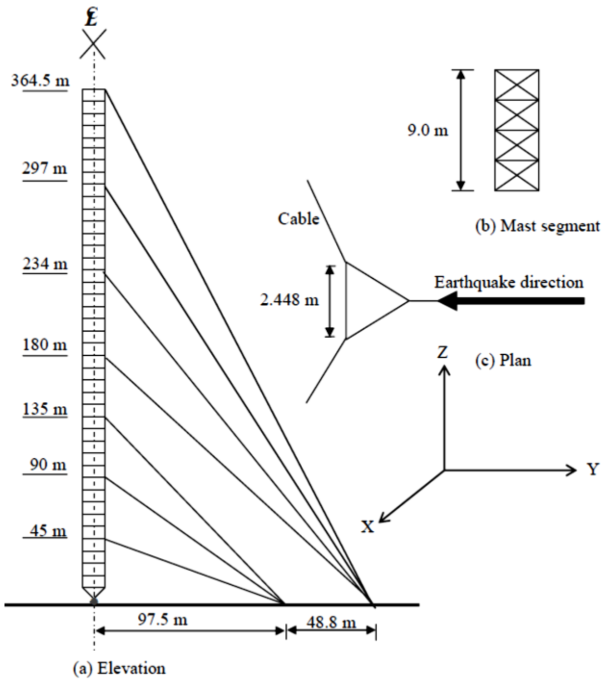


Fig. 4 Profile of the 364.5 m guyed tower, (a) Elevation, (b) Mast segment, and (c) Plan

presents the properties of the guys and mast members. For bracing configurations studied, the steel weight of the bracing was maintained constant and was determined based on that for the St. Andrew’s bracing configuration, Fig. 3 (a). Thus, all masts have the same mass distribution along the height. In this study, the analysis was carried out on the tower without accounting for the inertia effects of antenna mass for the sake of comparison.

2.2 Finite-element modeling

Based on previous parametric investigations (Ismail 2016, De Olivera *et al.* 2007, De Silva *et al.* 2002), a modelling strategy combining three-dimensional beam and truss finite elements was proposed. In this methodology, the legs of the tower mast used three-dimensional beam elements, while the bracing system utilized truss finite elements. The adoption of truss finite elements in the bracing system (diagonals and horizontal members) is explained by two main reasons: a single bolt indicating a hinged behavior usually makes the bracing system connections to the main structural system (i.e., legs of the mast). Additionally, the low flexure stiffness values, associated with the bracing elements that may be fully welded to the mast legs, imply that no significant moments will be present or transmitted to these structural members. Table 1 shows that for the 364.5 m guyed tower, the area of the horizontal and diagonal members are 6.4% and 36% of the area of the legs, respectively. This makes the flexural stiffness of the horizontal and diagonal members as 0.4% and 13% of that of the leg, respectively.

In this study, we used ABAQUS (version 6.2), the finite-element software program that has been used previously by many other researchers for the analysis of different types of structures, such as steel (for example, Wang *et al.* 2015), concrete (for example, Rama *et al.* 2017), composite (for example, Alizadeh and Dehestani 2015) and single-layer structures (for example, Lopez-Arancibia *et al.* 2015). In this study, a three-dimensional modeling was used to simulate the tower response. The diagonal and horizontal members were modeled using two-node three-dimensional truss elements (*T*3*D*2) with three degrees of freedom at each node. Two nodes three-dimensional frame elements (*B*31) were used to model the legs of the guyed mast. The base of the tower was modeled so that the translational degrees of freedom as well as the rotation about the axis along the height of the tower were prevented. Each individual guy was discretized into several elements according to its length. The *NO COMPRESSION* option in the ABAQUS software is used to account for the slackening of the guys. The interaction between the guys and the mast was considered as well as the pre-tension in the guys, which were initially pre-tensioned to 10% of their ultimate strength. Geometric nonlinearity option was considered in the static and the seismic analysis. This geometric nonlinearity is due to the large displacements of the mast under the effect of both horizontal and vertical loads. Since some of the guys are tighten while the others slacken during mast vibration, the stiffness contribution of the tightened guys could approach their elastic yield stiffness with no contribution from the slackened ones. This

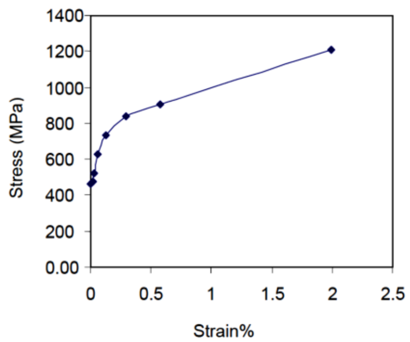


Fig. 5 Steel stress-strain relationship used in the static analysis

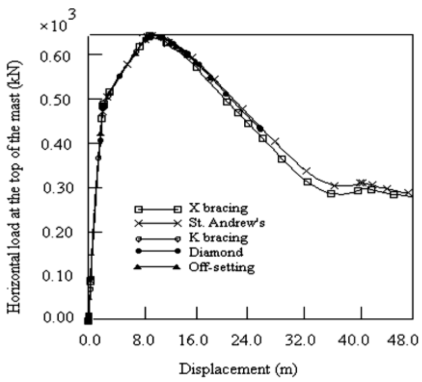


Fig. 6 Load-displacement relationship for the static load applied at the upper tip of the mast

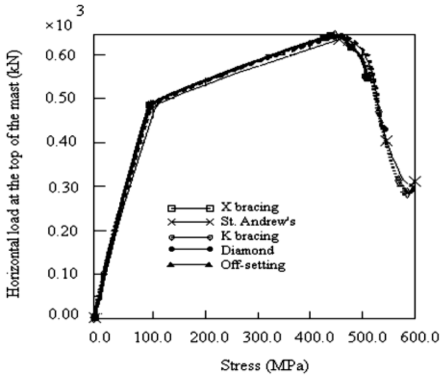


Fig. 7 Load-steel stress relationship in the static load applied at the upper tip of the mast

results in changes in the boundary conditions, during the motion of the tower. In addition, the catenary profiles of the guys will contribute to the non-linearity. For the static analysis, the material nonlinearity was also considered to determine the maximum horizontal load that can be carried at the upper tip of the mast.

2.3 Static lateral load-displacement response

A comparison was made between the different bracing configurations, shown in Fig. 3, for the tower subjected to increasing lateral static load at the top of the mast. The objective was to determine the maximum load that can be carried by the mast in the y-y direction, shown in Fig. 4, before global instability occurred. High strength steel with an ultimate stress of 1200 MPa was used for both the mast and the cables. The elastic-plastic stress-strain relationship of the steel is shown in Fig. 5.

The results of the analyses are presented in Fig. 6 showing the horizontal load-displacement response under increasing lateral static load at the top of the mast for different bracing configurations. From this plot, it is observed that the load–displacement relationship is not significantly influenced by the bracing configuration. Additionally, it is observed that strength degradation starts almost at the same load level when the horizontal force reaches about 644.0 kN and the displacement at the top of the mast is about 10.0 m. The steel maximum stress corresponding to this peak load capacity point is about 460 MPa as shown in Fig. 7. For all the bracing configurations, the steel maximum stress was developed at the second top cluster (i.e., exactly at 297 m from the base). This means that if these towers were designed to undergo a maximum displacement at the top of the mast not exceeding 1.0% of its height (i.e., 3.65 m), then all bracing configurations studied would satisfy this serviceability condition.

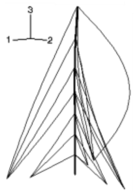
2.4 Free-vibration analysis

This section discusses modal analysis of the guyed towers. The determination of natural frequencies and mode shapes is important since it has a major influence on the dynamic response of the structure. This study involved determining bending and torsional mode shapes of the guyed towers.

Fig. 8 shows the first modes for the five bracing configurations considered in this study. It was found that the fundamental transverse mode of the longest guy cable governs the dynamic response of the tower irrespective of bracing configuration. This observation is expected since the cables were modeled to be the same in all the towers for the sake of comparison between different bracing configurations of the mast.

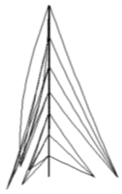
It was also observed that bending and torsional modes appeared in higher modes. The first bending and torsional modes for different towers are shown in Figs. 9 and 10, respectively. The results show that the tower with K bracing exhibits less stiffness than the other ones. Table 2 presents the values of the first three frequencies of the guy cables as well as the bending and torsional modes of the mast with various bracing configurations. It was observed that the difference in the first mode frequency of different models is small. This is because, for such flexible structures, the distribution of the mass is the same in all the towers considered in this study. This finding confims the previous conclusion from the static analysis that using different bracing configuration shows no significant effect on the response of the towers.

Accordingly, the choice of a specific bracing configuration for these tall guyed towers should be based on the simplicity of the configuration for assembly and erection in the field. Although Table 2 shows no significant differences in the frequency values for the first three modes of the Guy as well as slight differences in the bending and torsion modes of vibration of the mast, it is worth mentioning that the tower with K bracing configuration

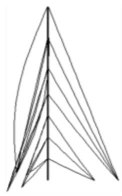


(a)

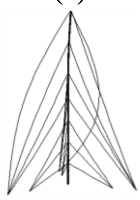
(b)



(c)



(d)

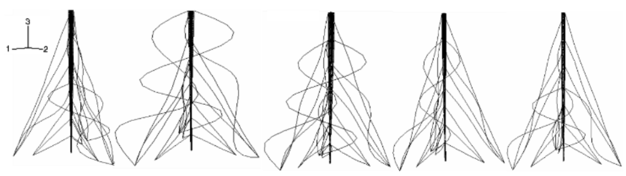


(e)

Fig. 8 First mode shape for the guy cable with bracing configuration of (a) St. Andrew's, (b) X, (c) K, (d) Diamond, and (e) Off-setting

(a) (b) (c) (d) (e)

Fig. 9 First bending mode shape, (a) St. Andrew's, (b) X, (c) K, (d) Diamond, and (e) Off-setting



(a) (b) (c) (d) (e)

Fig. 10 First torsional mode shape, (a) St. Andrew's, (b) X, (c) K, (d) Diamond, and (e) Off-setting

Table 2 The first three fundamental frequencies of the guyed mast for different bracing configurations

Mode	Frequency (Hz)				
	St. Andrew	K	X	Diamond	Off-setting
Guy mode 1	0.163	0.163	0.163	0.163	0.163
Guy mode 2	0.167	0.166	0.164	0.167	0.167
Guy mode 3	0.182	0.181	0.170	0.182	0.182
Bending mode 1	0.52	0.49	0.52	0.52	0.52
Bending mode 2	0.65	0.52	0.65	0.65	0.65
Bending mode 3	0.69	0.65	0.68	0.69	0.69
Torsional mode 1	0.55	0.49	0.56	0.58	0.56
Torsional mode 2	0.57	0.56	0.62	0.62	0.58
Torsional mode 3	0.61	0.57	0.65	0.63	0.62

showed the least frequency values among other studied bracing configurations for the first 3 bending, as well as torsion, modes of vibration for the same area of bracing members (a reduction between 1.7% and 25% based on the mode number and shape).

3. Seismic analysis: Effect of multi-support excitation

In guyed towers especially those with very tall masts, the distance between opposite anchors are particularly large such that using synchronous base motions at all anchors would not reflect the reality of ground motions. If a shear wave velocity of 2 to 3 km/s is assumed, a time delay ranging from 0.03 s for a 60 m tall tower to 0.29 s for a 591 m tall tower would exist between the ground motion inputs. These time gaps are likely to affect the dynamic response of guyed towers. Guevara and McClure (1993) dealt with the dynamic response of guyed towers under seismic loading considering the travel distance between the anchors. They analyzed two guyed towers (24 m tall with two guy levels and 107 m tall with six guy levels). Their results indicated the importance of the interaction between the guys and the mast, and also the fact that multiple support excitation of the tallest tower caused additional dynamic effects that cannot be presented when only synchronous ground motions are used. However, only two towers with the maximum height of 107 m were used in the study. A more comprehensive study is required to investigate the effect of time lag on the seismic response of such towers. In the present study, six towers of heights of 60 m, 120 m, 214 m, 283 m, 314 m, 364.5 m were used while assuming a shear wave velocity of 2 to 3 km/s.

3.1 Description of the towers

Table 3 shows the geometric data for these guyed towers of different height and radius of anchor set. It should be mentioned here that the average segment vertical span between each consecutive set of guy is 25 m ± 20% for the 60 m tower, 120 m tower and 214 m tower. However, it is 60 m ± 20% for the towers with the height of 283 m and

Table 3 Details of towers considered in multi support seismic excitation analysis

Tower	Height (m)	Number of guy levels	Panel width (m)	Panel height (m)	Total weight (kN)	Radius of anchor set No. 1 (m)	Radius of anchor set No. 2 (m)
1	60.0	3	0.61	0.76	17.5	48.77	-
2	120.0	4	0.91	1.10	33.70	30.00	60.00
3	214.0	7	1.52	1.52	237	73.10	170.70
4	283.5	4	3.00	3.00	2183	100.58	124.97
5	314.0	5	2.14	1.52	1244	125.00	213.40
6	364.5	7	2.45	2.25	2424	97.50	146.30

Table 4 Distribution of the guys along the mast as a percentage of the height

Tower (m)	Height at cable attachment points to the total height of the tower (h_i/h)							
	Lower zone			Medium zone		Top zone		
60.0	0.26			0.59				0.93
120.0	0.25			0.50	0.75			1.00
214.0	0.10	0.25	0.40	0.54		0.68	0.82	0.96
283.5	0.22			0.50	0.70			0.92
314.0	0.15		0.40	0.57		0.77		0.98
364.5	0.12	0.25	0.37	0.50		0.64	0.82	1.00

above. This is a typical average used in the telecommunication industry for most of guyed towers in this height range in North America. Table 4 shows the distribution of the guys along the height of the mast at the lower, medium, and the top zones of the mast. Note that the 60 m, 214 m, 283 m, and 314 m towers have free ends at the top. Furthermore, the difference in the distribution ratio of the cables over the height of the towers varies between 0 and 22% in the three zones except for the first cable for towers of height 60 m, 120 m, and 283 m. More details of these towers can be found elsewhere (Meshmesha 2005).

3.2 Finite-element modeling

The selected guyed towers in Table 3 were simulated by utilizing an equivalent beam-column model with two nodes and six degrees of freedom at each node. The equivalent beam-column analysis is based on the determination of the equivalent shear, torsion, and bending rigidities as well as the equivalent cross-sectional area of the mast (Meshmesha *et al.* 2003). This approach results in reduced computational time for the analyses by reducing the degrees of freedom. Two nodes three-dimensional frame elements (*B*31) were used to model the equivalent beam-column section of the guyed mast. The base of the tower and the guys were modeled as mentioned earlier. Fig. 11 shows views of the finite-element models of the six towers developed in this study.

3.3 Seismic analysis

ABAQUS software was used to investigate the seismic

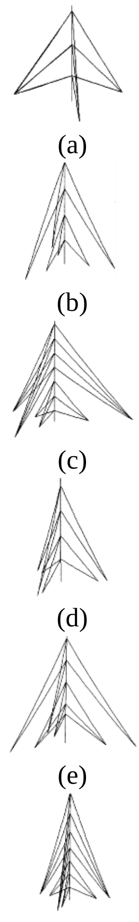


Fig. 11 Views of the finite-element models of the: (a) 60 m tower; (b) 120 m; (c) 214 m; (d) 283 m; (e) 314 m; and (f) 364 m tower

response of the towers. To show the effect of the travel distance on the response of the guyed towers, Saguenay Earthquake record of 1988, which is a 20-second event and of magnitude of 5.9, was used. The towers of different heights (from 60 m to 364.5 m) were subjected to three components of the earthquake record both synchronously and asynchronously. The distribution of shear force and bending moment along the mast as well as the displacement at the top of the mast, the maximum tensile stress in the cables, and the reaction at the base of the mast were determined. The earthquake input was applied at the base of the mast and at the anchors. A time delay ranging from 0.03 s for the 60 m tower to 0.1 s for the 364.5 m tower was considered. The prescribed accelerations of the earthquake components were applied at the base nodes in the three major axes of x, y, and z with the y-y direction as the main horizontal component. Figs. 12 and 13 show the acceleration time histories and the spectral acceleration for the main horizontal component of the selected earthquake record, respectively.

Nonlinear dynamic analysis in ABAQUS was carried out to account for the geometric nonlinearity in the structure. Implicit integration using the Hilber-Hughes-Taylor method (Hibbitt *et al.* 2000) was employed to solve the nonlinear problem. In the implicit integration, 100

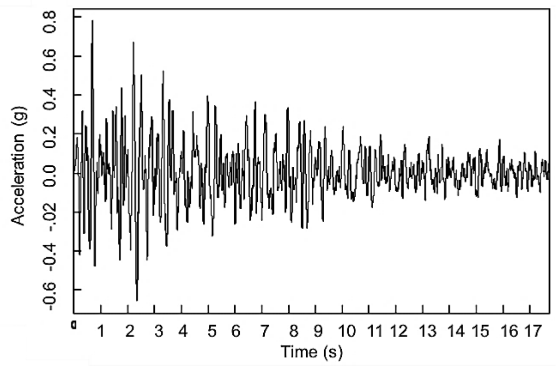


Fig. 12 Time history for the acceleration of Saguenay earthquake record

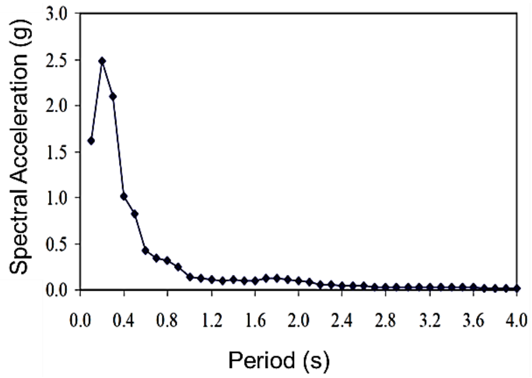
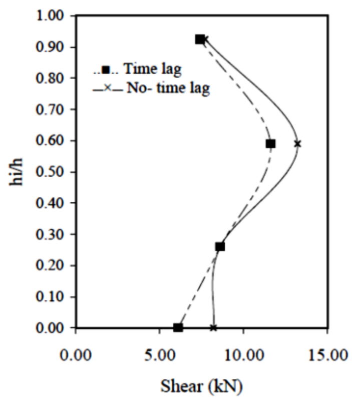
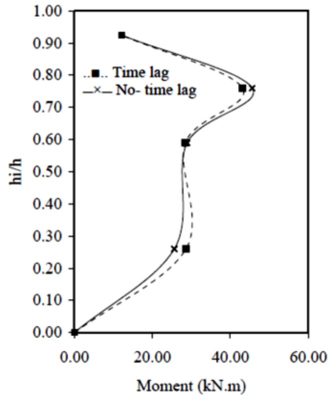


Fig. 13 Spectral acceleration of Saguenay earthquake record

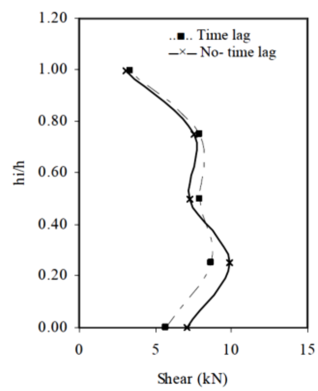


(a)

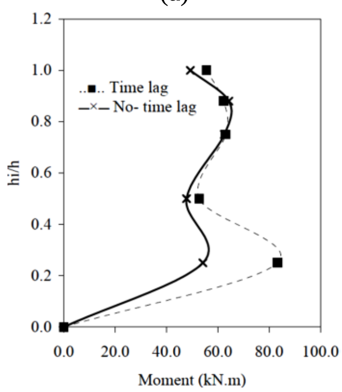


(b)

Fig. 14 For the 60 m tower: The distribution of (a) shear and (b) moment along the mast



(a)



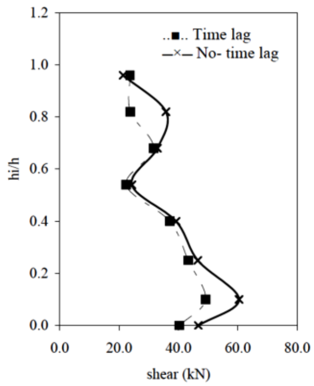
(b)

Fig. 15 For the 120 m tower: The distribution of (a) shear and (b) moment along the mast

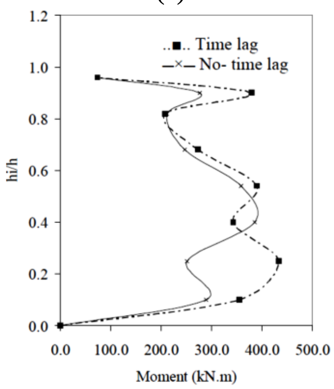
increments in each second were assigned with an incremental time of 0.01 second, which was sufficiently small to closely approximate the earthquake loading. In this study, it was decided to rely on the numerical damping for the difficulties associated with realistic modeling for cable and mast damping. It is the author's view that the accurate value of damping would not have significant influence on the responses for flexible structures such as guyed towers. However, a sensitivity study was conducted to choose the controller parameter *ALPHA*, where a value of -0.15 was chosen. The solution in ABAQUS involves using the large displacement theory and iteration procedures to reach a solution for the nonlinear problem.

The distribution of shear and bending moment at the critical sections of the mast in the studied towers are shown in Figs. 14 to 18. These figures illustrate the maximum shear force and bending moment responses of each tower versus hi/h, defined as the height at a given section to the total height of the tower. It can be observed that the inclusion of time lag does not show a general trend in the maximum bending moment distribution. For example, the maximum bending moment in the 60 m, 283 m, and 364.5 m towers decreases with the inclusion of the time lag while it increases for the 120 m, 214 m, and 314 m towers.

The bending moment distribution for the mast can be significantly different with the multi-support seismic excitation (i.e., when the time lag is included in the analysis). For example, the bending moments at the levels of the bottom and top guy are increased by 23% and 39%, respectively, when considering the multi-support excitation

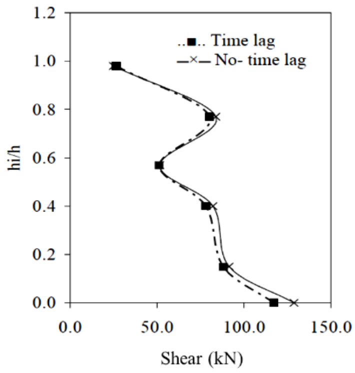


(a)

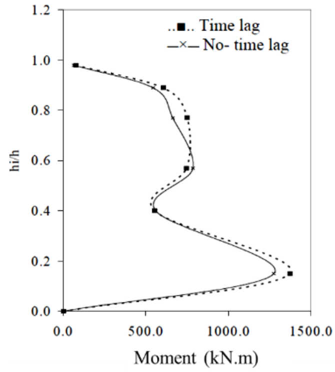


(b)

Fig. 16 For the 214 m tower: The distribution of (a) shear and (b) moment along the mast

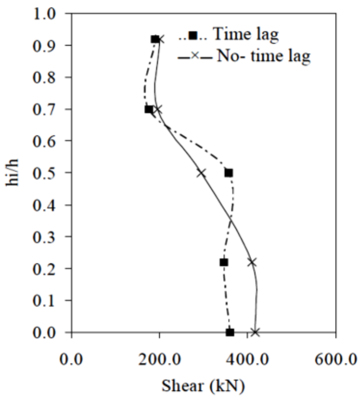


(a)

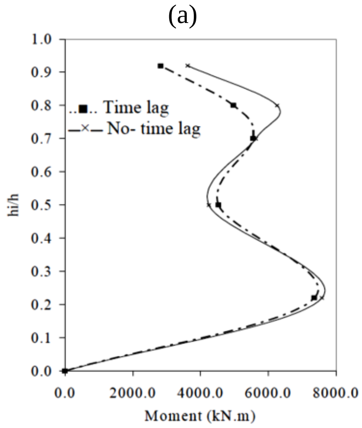


(b)

Fig. 18 For the 314 m tower: The distribution of (a) shear and (b) moment along the mast

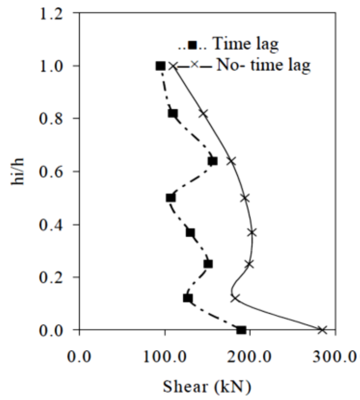


(a)

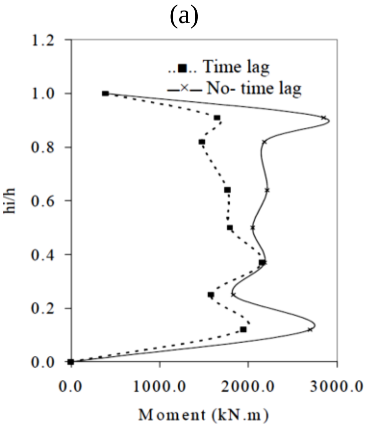


(b)

Fig. 17 For the 283 m tower: The distribution of (a) shear and (b) moment along the mast



(a)



(b)

Fig. 19 For the 364 m tower: The distribution of (a) shear and (b) moment along the mast

Table 5 Maximum responses for the 364.5 m tower

	Lateral Displacement (m)	Base Shear (kN)	Vertical Reaction (kN)	Max. Moment (kN.m)	Max. Tensile Stress (MPa)
Time lag	0.51	151.7	9,742	2,155	221.0
No-time lag	0.56	230.9	10,210	2,846	285.2
Difference %	8.0	34.3	4.8	32.1	29.0

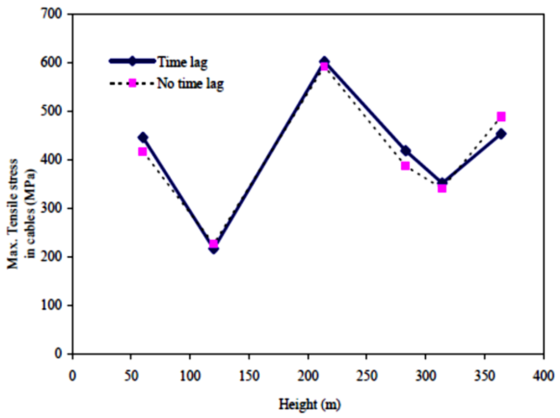


Fig. 20 Maximum tensile stress values in cables

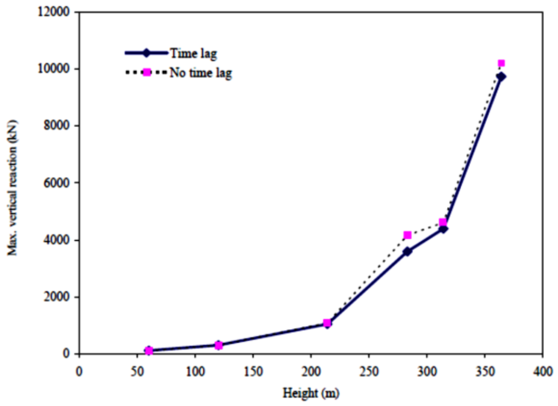


Fig. 21 Maximum vertical reactions

in the 214 m tall tower. However, the bending moments in the mast at the levels of the bottom and top guy of the 364.5 m tall tower are decreased by 34% and 41%, respectively, with the multi-support excitation.

Based on the results presented in Figs. 14 to 19, the maximum base shear decreases when the time lag is considered in the analysis irrespective of the tower height. It is also observed that the maximum shear over the length of the mast is lower when the time lag is accounted for in the analysis.

The maximum tensile stresses in the cables for each tower as well as the maximum vertical reactions are shown in Figs. 20 and 21, respectively. Results show an insignificant effect of the inclusion of the time lag on the structural responses of the cables and the vertical reactions. For towers of heights of 214 m and more, the effect of including the time lag in the analysis is slightly tangible. This effect for taller towers is expected because, in taller towers, the time lag is more likely to change the intensity and the frequency content of the input ground motion.

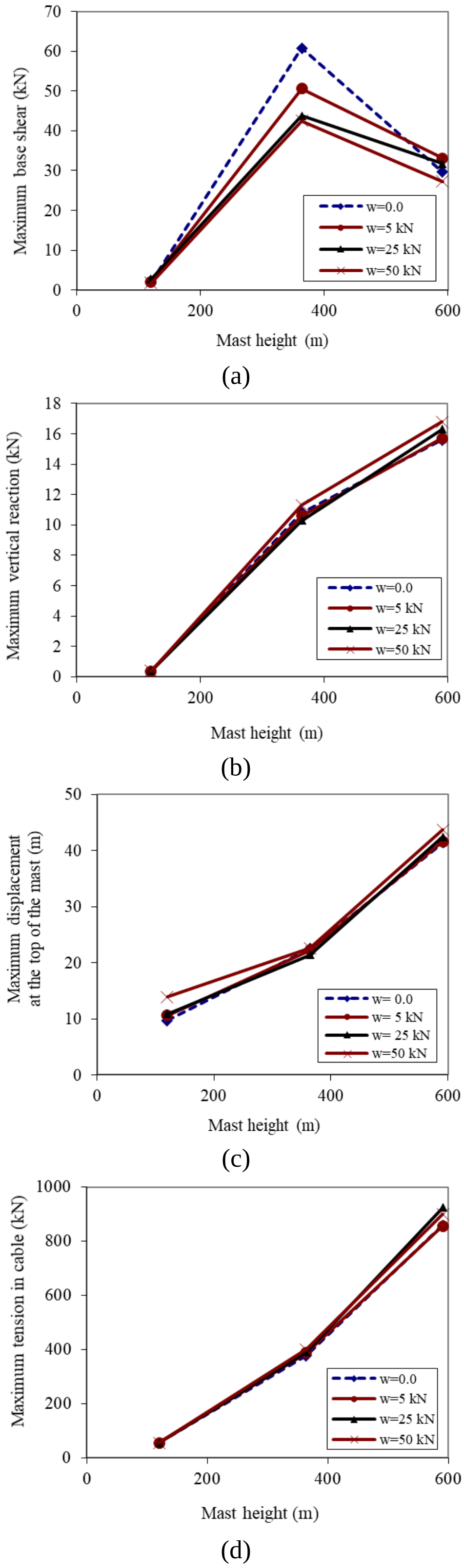


Fig. 22 Effect of antenna weight on: (a) maximum base shear; (b) maximum total displacement at the top of the mast; (c) maximum vertical base reaction; and (d) maximum tension in the outermost cable

Table 5 summarizes specific results for the 364.5 m tower as an example. It can be observed that for this tower, the inclusion of multiple support excitations decreases the total lateral displacement by 8%, the base shear by 34.3%, the vertical reaction by 4.8%, the maximum moment in the mast by 32.1% and the tensile stress in the cables by 29%.

4.4 Effect of antenna weight

A sensitivity analysis for three towers of 120 m, 364 m and 591 m tall was carried out using different antenna weights at the top of the mast, which is the most common location for the antenna. Depending on their types, the antenna weight varies. For example, the weight of the dishes ranges from 3.8 kN to 5.6 kN, whereas the weight of the top and side mounts ranges from 25.0 kN to 55.0 kN. The latter weights are often used in towers of taller than 200 m.

For the sensitivity analysis, three components of the 1940 El Centro earthquake record with a peak ground acceleration of 0.35 g were used. Fig. 22 show the effect of antenna weight on the maximum base shear, maximum displacement at the top of the mast, maximum vertical reaction at the base, and maximum tension force in the outermost cable. These results demonstrate that antenna weight generally does not significantly influence the response, except for the maximum base shear of the 364 m tower, where the base shear is reduced by 30% with the increased antenna weight from zero to 50 kN.

4. Conclusions

The first part of this paper studies the effect of using different bracing configurations on the static and dynamic response of guyed towers. The second part of the paper deals with the delay time of applied seismic excitation; it examines the effect of earthquake travel distance on the seismic response of guyed towers of different heights. When different structural supports are excited by different or delayed signals, this leads to complex dynamics, and can have particular significance on the response of structures with large distances between their supports. Finally, the effect of antenna weight on the seismic response of guyed towers is studied. Based on the results obtained from this study conducted on selected guyed towers, the following conclusions can be made:

For the guyed towers with five bracing configurations examined in this study, the type of bracing configuration showed insignificant influence on the static and dynamic responses. Although slight differences in the bending and torsion modes of vibration of the mast were observed, it is worth mentioning that the tower with K bracing configuration showed the least frequency values among other studied bracing configurations for the first three bending, as well as torsion, modes of vibration for the same area of bracing members. As a result, the choice of the bracing configuration should be based on economy with respect to production, transportation and erection. This conclusion is limited to the height, weight and geometric configurations of the studied tower.

• The inclusion of the time lag in the seismic analysis of guyed towers is likely to change the intensity and the frequency content of the input ground motion. Accordingly, the responses of theses towers will significantly be affected when including theses time lags in the analysis.

• The results show insignificant influence of the antenna weight on the seismic response of the towers, with possible reduction in base shear. Depending on the mast height, an increased antenna weight can reduce the tower maximum base shear while other response quantities, such as cables tension force, are found to be insensitive to variation in the antenna weight.

Acknowledgments

The financial support from the Natural Science and Engineering Research Council of Canada (NSERC) is greatly appreciated.

References

Alizadeh, E. and Dehestani, M. (2015), “Analysis of composite girders with hybrid GFRP hat-shape sections and concrete slab”, *Struct. Eng. Mech.*, **54**(6), 1135-1152.

Amiri, G. (2002), “Seismic sensitivity indicators for tall guyed telecommunication towers”, *Comput. Struct.*, **80**(3-4), 349-364.

ASCE (2001), *Dynamic Response of Lattice Towers and Guyed Masts*, Report Prepared by the Task Committee on the Dynamic Response of Latticed Towers, Technical Committee on Spatial Structures, Structural Engineering Institute, American Society of Civil Engineering, Reston, Virginia, U.S.A.

Battista, R.C., Pfeil, M.S., Carvalho, E.M. and Varela, W.D. (2018), “Double controller of wind induced bending oscillations in telecom towers”, *Smart Struct. Syst.*, **21**(1), 99-111.

BSI (1994), *British Standard: Lattice Towers and Masts: Part 4, Code of Practice for Lattice Masts*, BS 8100 Part 4, British Standards Institution, London, U.K.

Canadian Standard Association (2015), *National Building Code of Canada*, Rexdale, Ontario, Canada.

Canadian Standard Association (2013), *Antenna Towers and Antenna Supporting Structures*, CSA-S37-13, Rexdale, Ontario, Canada.

Da Silva, J., Vellasco, P., Andrade, S. and Oliveira, M. (2002), “An evaluation of structural steel design systems for transmission and telecommunication towers”, *Proceedings of the International IASS Symposium*, Warsaw, Poland.

De Oliveira, M., Da Silva, J., Da S., Vellasco, P., De Andrade, S. and De Lima, L. (2007), “Structural analysis of guyed steel telecommunication towers for radio antennas”, *J. Brazil. Soc. Mech. Sci. Eng.*, **29**(2), 185-195.

De Macedo, R. (2016), “Seismic response of transmission line guyed towers with and without the interaction of tower conductor coupling”, M.Sc. Dissertation, Concordia University, Canada.

Desai, Y. and Punde, S. (2011), “Simple model for dynamic analysis of cable supported structures”, *Eng. Struct.*, **23**(3), 271-279.

Electric Power Research Institute-EPRI (2009), *Transmission Line Reference Book: Wind-Induced Conductor Motion*.

Ellis, S., Shyu, T. and Quenneville, H. (1998), “Latticed mast structures”, *ASCE J. Struct. Eng.*, **124**(2), 203-206.

Faridafshin, F. and McClure, G. (2008), “Seismic response of tall guyed masts to asynchronous multiple-support and vertical ground motions”, *ASCE J. Struct. Eng.*, **134**(8), 1374-1382.

Federal Emergency Management Agency-FEMA (1991), *Earthquake Resistant Construction of Electric Transmission and Telecommunication Facilities Serving the Federal Government Report*, Earthquake Hazard Reduction Series 56.

Guevara, E. and McClure, G. (1993), "Nonlinear seismic response of antenna-supporting structures", *Comput*. *Struct*., **47**(4-5), 711-724.

Hensley, G. (2005), "Finite element analysis of the seismic behavior of guyed masts", M.Sc. Dissertation, Virginia Polytechnic Institute and State University, U.S.A.

Hensley, G. and Plaut, R. (2007), "Three-dimensional analysis of the seismic response of guyed masts", *Eng*. *Struct*., **29**(9), 2254-2261.

Hibbitt, D., Karlsson, I. and Sorenson, J. (2000), *ABAQUS User's Manual Version 6.2*, Hibbitt, Karlsson and Sorenson Inc., Providence, RI.

Ismail, A. (2016), "Seismic assessment of guyed towers: A case study combining field measurements and pushover analysis", *HBRC J. Hous. Build. Nat. Res. Cent*., **12**, 47-53.

Khan, M.A., Siddiqui, N.A. and Abbas, H. (2004), "Reliability analysis of latticed steel towers against wind induced displacement", *Steel Compos*. *Struct*., **4**(1), 9-21.

Lacarbonara, W. and Ballerini, S. (2009), "Vibration mitigation of guyed masts via tuned pendulum dampers", *Struct*. *Eng*. *Mech*., **32**(4), 517-529.

Law, S.S., Bu, J.Q., Zhu, X.Q. and Chan, S.L. (2006). "Response prediction of a 50 m guyed mast under typhoon conditions", *Wind Struct*., **9**(5), 397-412.

Li, Z.J., Li, P., He, Z. and Cao, P. (2013), "Static and free vibration analysis of shallow sagging inclined cables", *Struct*. *Eng*. *Mech*., **45**(2), 145-157.

Lopez-Arancibia, A., Altuna-Zugasti, A.M., Aldasoro, H.A. and Pradera-Mallabiabarrena, A. (2015), "Bolted joints for single-layer structures: Numerical analysis of the bending behavior", *Struct*. *Eng*. *Mech*., **56**(3), 355-367.

Madugula, M.K. (2001), *Dynamic Response of Lattice Towers and Guyed Masts*, Prepared by the Task Committee on the Dynamic Response of Lattice Towers of the Technical Committee on Special Structures and the Technical Administrative Committee on Metals of the Structural Engineering Institute of ASCE, SEI-Structural Engineering Institute.

Mahboba, M., Mahmuda, J., MohdAzlanb, M. and Jumahata, A. (2013), "Finite element analysis of telecommunication mini mast structure", *Proc*. *Eng*., **68**, 258-264.

Meshmesha, H. (2005), "Dynamic analysis of guyed antenna towers for seismic loads", Ph.D. Dissertation, University of Windsor, Windsor, Ontario, Canada.

Meshmesha, H., Sennah, K. and Kennedy, J. (2003), "Beam-column analysis of guyed musts", *Proceedings of the 31st Annual Conference of the Canadian Society of Civil Engineering*, Moncton, New Brunswick, Canada.

Meshmesha, H., Sennah, K. and Kennedy, J.B. (2006), "Simple method for static and dynamic analyses of guyed towers", *Struct*. *Eng*. *Mech*., **23**(6), 635-649.

Oskoei, S.A.G. and McClure, G. (2011), "A novel approach to evaluate the equivalent dynamic stiffness of guy clusters in telecommunication masts underground excitation", *Eng*. *Struct*., **33**(5), 1764-1772.

Oskoei, S. and McClure, G. (2012), "A new robust linearized seismic analysis method for tall guyed telecommunication masts", *ASCE J*. *Struct*. *Eng*., **138**, 502-513.

Pratt, M. (1988), "Elastic non-linear study of latticed frame structures", M.Eng. Dissertation, Royal Military College of Canada, Kingston, Canada.

Rama, J.S., Chauhan, D.R., Sivakumar, M.V.N., Vasan, A. and Murthy, A.R. (2017), "Fracture properties of concrete using damaged plasticity model-a parametric study", *Struct*. *Eng*. *Mech*., **64**(1), 59-69.

Rodrigo, F.D.M. (2016), "Seismic response of transmission line guyed towers without and with the interaction of tower conductor coupling", M.Sc. Dissertation, Concordia University, Montreal, Canada.

Shi, H. and Salim, H. (2015), "Geometric nonlinear static and dynamic analysis of guyed towers using fully nonlinear element formulations", *Eng*. *Struct*., **99**, 492-501.

Sparling, B.F. and Wegner, L.D. (2007), "Estimating peak wind load effects in guyed masts", *Wind Struct*., **10**(4), 347-366.

Telecommunications Industry Association (TIA) TR-14 (2017), *Communication and Small Wind Turbine Support Structures Committee Today Published Revision H of the ANSI/TIA-222 Structural Standard for Antenna Supporting Structures*, Antennas and Small Wind Turbine Support Structures.

Wang, M., Shi, Y. and Wang, Y. (2015), "Application of steel equivalent constitutive model for predicting seismic behavior of steel frame", *Steel Compos*. *Struct*., **19**(5), 1055-1075.

Wu, Q., Takahashi, K. and Chen, B. (2007), "Influence of cable loosening on nonlinear parametric vibrations of inclined cables", *Struct*. *Eng*. *Mech*., **25**(2), 219-237.

Wu, G., Zhai, C., Li, S. and Xie, L. (2014), "Effects of near-fault ground motions and equivalent pulses on large crossing transmission tower-line system", *Eng*. *Struct*., **77**, 161-169.

Xia, G.Y. and Cai, C.S. (2011), "Equivalent stiffness method for nonlinear analysis of stay cables", *Struct*. *Eng*. *Mech*., **39**(5), 661-667.

Yang, S.C. and Hong, H.P. (2016), "Nonlinear inelastic responses of transmission tower-line system under downburst wind", *Eng*. *Struct*., **123**, 490-500.

American Society of Civil Engineers-ASCE/SEI (10-15), *Design of Lattice Steel Transmission Structures*.

CC