# PARAMETRIC STUDIES OF GUYED TOWERS UNDER WIND AND SEISMIC LOADS

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**Abstract.** As the wireless communications spread there is an increasing demand of antenna support-ing structures. A very common systems are lattice towers, either self-supported or guyed. The latter are chosen for economical reasons when there is enough space for their location. Radio and television employ structures that range between 100 and 600 m and communication towers for mobile phones are around 60 m though higher structures are also constructed. For large heights, guyed masts are in-dicated. However, it depends on the customer preferences, suppliers, budget and location. Generally, self-supported structures are preferred in urban areas and guyed masts in the countryside. Nowadays the demand for more accurate and reliable communication systems poses higher structural demands since the signal technology sometimes requires of very small motions of the supporting structures to achieve a high quality transmission. The design of these structures is, in general, carried out following the standard codes and simplified models. The dynamic actions, as wind and earthquakes, are not addressed in detail with exception of special cases, despite the large potential of adverse impact. One of the parameters significant to the dynamic response of the structure is the guy pre-load value. Although there is a recom-mended pre-stress of about 10 % of the ultimate stress, it is frequently found that the guys are under or overloaded, situation that can change the dynamic response of the whole structural system. In this work, a parametric study on the effect of the guy pre-load is carried out. A typical guyed tower is analyzed with a finite element model in which the lattice mast is represented by an equivalent beam-column element and the guys (or cables) by truss prestressed elements. The wind load is calculated from the design code regarding the height distribution and roughness characteristics. To account for the dynamic loads, a sim-plified approach is followed. The frequency content of the peak region of a typical longitudinal velocity spectrum is used to construct a temporal function. Various values of pre-loads are considered for the guys. The dynamic response is referenced to the quasi-static one and conclusions regarding the influence of the level of prestress are drawn. Additionally, analogous studies on the same tower with seismic loads are reported. It is observed that the influence of the guy pre-load variation is diverse depending on the type of dynamic load.

# 1    INTRODUCTION

For many years guyed masts have been used to support antennas for radio, tv and other communications. This structure have clear advantages in the open country, where there are no restrictions on the position of the cable anchors. However, this kind of structures is, sometimes, also found in urban areas, due to its low cost, compared with other tipologies. A typical configuration comprises a lattice tower with triangular cross-section (three legs, horizontal and diagonal members) (see Figure 1). The height is variable depending on the functions, but nowadays is not exceptional to see 300 m-height towers. The main structural characteristics are the large slenderness of the mast and several levels of taut guys. Dynamic loads as wind or earthquake, are simplified as quasistatic loads that represents the mean of the dynamic phenomena amplified with factors that account for the dynamics characteristics at each case, following standard codes and recomendations. Since the wind structure is turbulent and the masts are flexible and sensitive to dynamic loads, the dynamic response becomes important in the analysis of guyed masts. The mast acts strongly in a non-linear fashion when the guys vary from a slack to a taut state.



Figure 1: Typical guyed tower.

Research on this subject includes works by Kahla (1993) that employs equivalent beam methods in order to simplify five lattice mast and carries out statics analysis.

Wahba et al. (1998) and Wahba and Monforton (1998) evaluate the behavior of guyed towers, modeling the mast as a lattice truss beam or with beam elements. The finite elements procedure is used to model six existing guyed masts, in order to study the influence of ic accretion, guy initial tensions and torsion resistors on the dynamic response of the structure.

Kewaisy (2001) proposes a hybrid model that includes non-linear considerations, modeling the guys with a finite difference approximation and the mast with finite elements.

Kahla (2000) analyzes the response of a guyed mast to a guy rupture under no wind pressure, using a program developed by the author.

Punde (2001) studies dynamics of cable supported structures using a generalized finite element approach.

The non linear spectral element method in order to analyze the non linear dynamic response of a guyed mast is used by Horr (2004)

The dynamic response of guyed masts using different models for cables and evaluates the variation of the stiffness of the complete system using different levels of pre-stress on guys is compared by Preidikman et al. (2006)

Meshmesha et al. (2006) introduce an equivalent beam-column analysis based on an equivalent thin plate approach for lattice structures, then evaluates the accuracy of the proposed method and clasic methods to determine the equivalent beam propieties (the unit load method and the energy approach) in determining the response of a guyed tower subjected to static and seismic loading.

The finite elements approach was also used by Shi (2007) and de Oliveira et al. (2007).

Lu et al. (2010) introduce the principle of harmonic wave superimpose method for wind velocity simulation, as well as the improved method by introducing FFT in harmonic superimpose wave method. Wind velocity time series along the height of guyed-mast was simulated with the improved method.

Matuszkiewicz (2011) evaluates selected problems concerning designing of guyed masts with lattice shaft in accordance with the "EN 1993-3-1: Design of steel structures. Part 3-1: Towers, masts and chimneys-Towers and masts" and discusses the method of application of the mast shaft geometrical imperfections in the calculation.

In this work, a simplified model of a 120 m guyed tower is studied. Its dynamic response, under the action of wind and earthquake loads, with five different pre-stress on the cables, is tackled through a finite element procedure. The lattice mast is modeled through an equivalent beam-column. Then, the finite element model of the mast is made of two-node beam elements and is supposed fixed at the base. Four levels of cables are modeled with truss elements considering the lack of compression capability of the cables. To account for the intrinsic nonlinear behavior of this structural system, the analysis is carried out with a mechanical event simulation modulus of the finite element package ALGOR-Inc. (2009) for wind studies and SAP2000 (2007) for earthquake analyses.

In the wind loaded model, the gravity loads on the tower and cables and the initial pre-stressing of the cables first are applied until the model attains static equilibrium (avoiding the transient perturbation). Then the load, either static (designed as specified in codes) or dynamic, is activated. The frequency content of the peak region of a typical longitudinal velocity spectrum is used to construct a temporal function. The dynamic response is referenced to the quasi-static one and conclusions regarding the influence of the level of prestress are drawn. This simplified approach does not account for the spatial correlation. The authors currently are studying a wind load model that includes these effects Castro et al. (2007).

The earthquake loaded model includes cable elements instead truss elements for guys. Four seismic accelerograms (San Juan, 23-11-1977; Mendoza, 26-1-1985; Valparaiso, 03-03-1985 and Northridge, 17-1-1994) were used for loading, in order to observe the dynamic response of the guyed mast with earthquakes with differents characteristics. Both San Juan and Valparaiso accelerograms are vibratory type earthquakes, while in Mendoza and Northridge have the characteristics of impulsive earthquakes.

The wind study is described in detail and the earthquake study results (already reported at Guzmán et al. (2010)) are briefly discussed and presented for the sake of comparision. It is observed that the influence of the guy pre-load variation is diverse depending on the type of dynamic load.

## 2   WIND LOADED MODEL

Here, the finite element model considered is similar to that used by Punde (2001). The three dimensional guyed tower is 120 m high and has four guy levels separated by 30 m, with three guys at each level, oriented in vertical planes separated by $120^o$ , and two sets of guy anchors in each of the three planes. (see Figure 2)

The mast is modeled as a beam-column, made of 12 6-DOF beam elements and it is fixed at the base. Each guy is modeled using 20 3-DOF two-node pre-stressed truss elements. To find the dynamic non-linear response the finite element software ALGOR-Inc. (2009), that allows consideration of large displacements, was used.
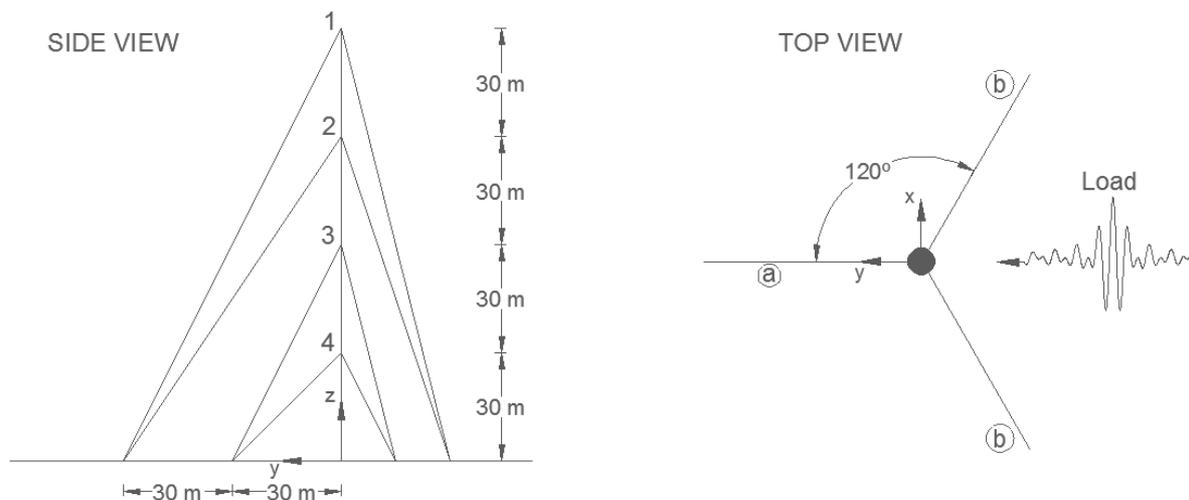


Figure 2: Guyed mast geometry.

The mast weight per unit length is 61 kg/m, the modulus of elasticity is 209 GPa, the second area moment in any direction is 0.0018 $m^4$ and the cross-sectional area is 0.00198 $m^2$. The guy cross-sectional area is 0.0002 $m^2$, the modulus of elasticity is 150 GPa, the weight per unit lenght is 2.55 kg/m and the referent case pretension is 25 kN. However the model was also calculated with 15, 20, 30 and 35 kN to evaluate the influence of this parameter on the dynamic response.

### 2.1   Loads

In order to simulate typical load conditions, three loads were taken into account: weight (gravitational load), pre-stress on the cables and dynamic and static loads, in the $y$ direction (which defines an axis of symmetry in the arrangement of cables, see Fig. 2). Only displacements on the $y$ direction will be reported. At this stage no loads are applied on guys.

#### 2.1.1   Static Wind Load

The static wind load was designed following the argentine standard code CIRSOC-INTI (2005), which defines the static wind load as

$$F = q_z * G * C_f * A_f \tag{1}$$

where $F$ is the magnitude of the wind load, $G$ is the gust coefficient, which takes into account the effects of the dynamic amplification (resonance) and lack of correlation of loads, $A_f$ is the

exposed area of the mast, projected onto the plane normal to the loads and $C_f$ is a coefficient wich takes into account the shape of the structure, in this case, the mast. Its formula is:

$$C_f = 3,4 * \epsilon^2 - 4,7 * \epsilon + 3,4 \tag{2}$$

where $\epsilon = A_f/A_t$ and $A_t$ is the exposed area of the mast without holes.

In Formula 1 $q_z$ is the wind pressure and is formula is:

$$q_z = 0,613 * k_z * k_{zt} * k_d * V^2 * I \tag{3}$$

where $I$ defines the category of the structure, $V$ is the reference velocity, defined at each zone of the country (in this case, Bahía Blanca, Argentina), $k_{zt}$ is the topographic coefficient, $k_d$ is the direction coefficient, that takes into account the type of structure (i.e. lattice towers, buildings, etc.) and $k_z$ is a empirical coefficient that considers the load variation with height and is equation is:

$$k_z = 2.01 * (z/z_g)^{2/\alpha} \tag{4}$$

where $z$ is the height of the point considered, $z_g$ and $\alpha$ are obtained from code tables. Table 1 shows the numerical values adopted and calculated for the coefficients.

| Coefficient | $G$ | $A_f$ | $A_t$ | $C_f$ | $I$ | $V$ | $k_d$ | $k_{zt}$ |
|---|---|---|---|---|---|---|---|---|
| Value | 1.00 | 0.57 m$^2$ | 9.41 m$^2$ | 3.13 | 1.00 | 55 m/s | 0.85 | 1 |

Table 1: Coefficients for calculating static wind loads according to CIRSOC-INTI (2005)

Figure 3 shows the load variation with altitude and the values of all the nodal forces on the mast.
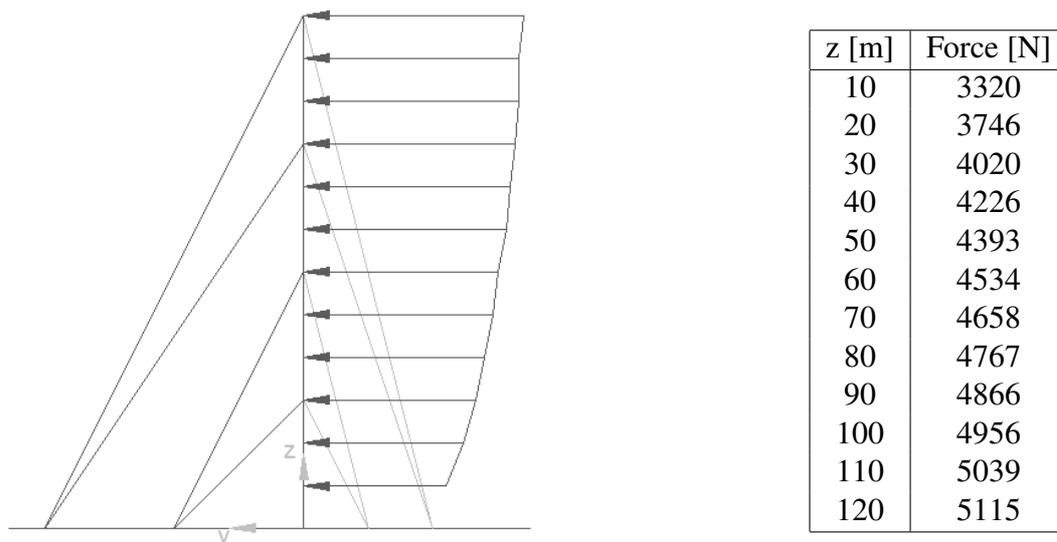


| z [m] | Force [N] |
|---|---|
| 10 | 3320 |
| 20 | 3746 |
| 30 | 4020 |
| 40 | 4226 |
| 50 | 4393 |
| 60 | 4534 |
| 70 | 4658 |
| 80 | 4767 |
| 90 | 4866 |
| 100 | 4956 |
| 110 | 5039 |
| 120 | 5115 |

Figure 3: Distribution of load in height

### 2.1.2 Dynamic Wind Load

The variation on height and mean value of the dynamic wind load was set using the same above procedure. The temporal variation function was added as a sum of cosine functions, where the employed frequencies were extracted from the peak zone of the Davenport's longitudinal turbulence power-spectrum density function (shown in Fig. 4, see e.g. Dyrbye and Hansen (1994)). The frequencies vary slightly with height (following the variation of the reference wind velocity with height), then the time variation of each nodal load is slightly different.
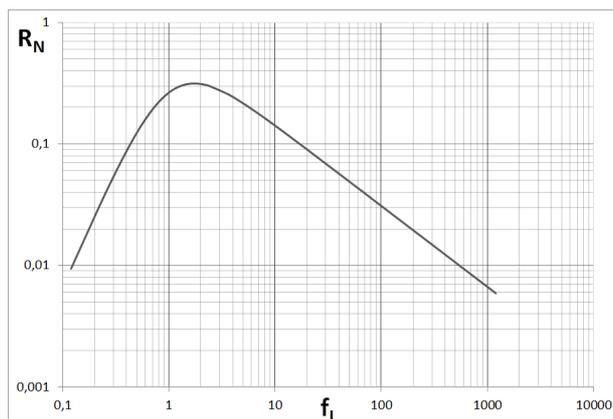


Figure 4: Power-spectrum density function used on designing the wind load

It was adopted 0.85 instead 1 for $G$ (the gust factor), considering that the dynamics effects will be observed with the addition of the time variation on the load. A 40% of the mean value of the nodal force was taken as the maximum amplitude of the introduced dynamic. Figure 5 illustrates generic shape of the time variation function.
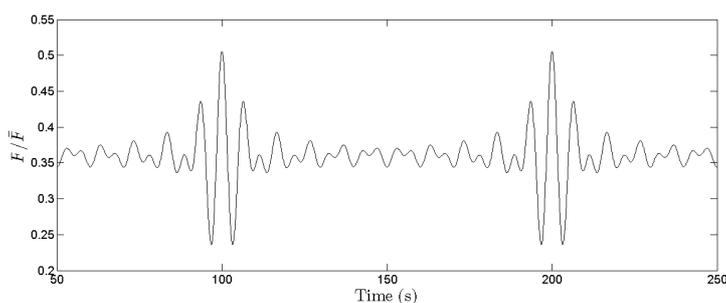


$$F = \bar{F} * (1 + 0.4 * (cos(\omega_1 * t) + cos(\omega_2 * t) + \cdots + cos(\omega_n * t)))$$

Figure 5: Typical time variation for loads

### 2.1.3 Load Application

In this study three loads were taken into account: gravity, pretension on the guys and the external excitation. The pretension is applied at the beginning and holds during the whole calculus. This value is added or subtracted to the temporal variations of the tension due the

external excitation. The gravity grows linearly from the beginning to the standard value in 3 s, and remains during the whole experiment. The wind load is applied starting at 4 s to the end of the calculus, to avoid the numerical instabilities due to the sudden applicattion of pretension. Figure 6 illustrates the time application pattern of the loads.
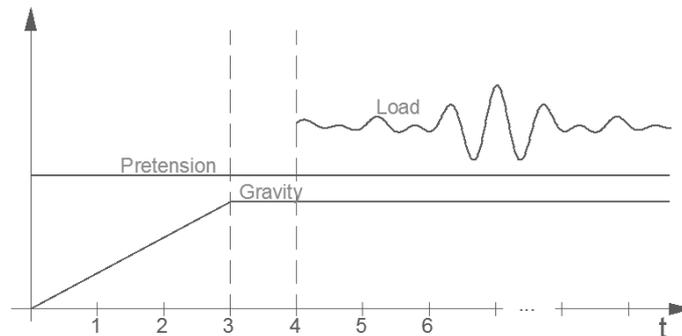


Figure 6: Temporal load application

The total time of calculus was 400 s and the $CR$ (capture rate) was 10 frames per second. Anyway, the time step is adaptive (is adopted by the software in order to find convergence). The $CR$ may seems large, but the fundamental period of the mast is 2.66 s and the wind loads periods are around 7 s, then the adopted value provide adequate precision to observe the dynamics of the problem.

## 2.2 Wind Study Results

### 2.2.1 Displacements

The influence of different values of pretension on the dynamic displacements was evaluated on the top of the mast.
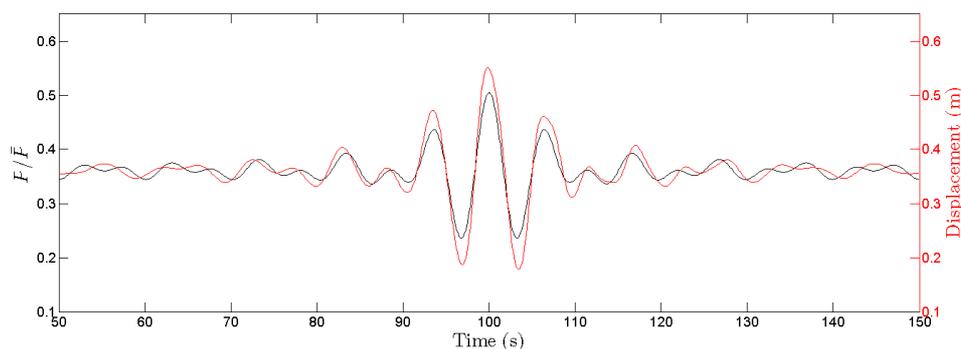


Figure 7: Typical displacement diagram.

In all the evaluated pretension cases, the dynamic displacements showed the same shape. The displacement curve follows the load curve, as can be seen on Figure 7. The complex non linear interaction between guys, mast and load may causes the differences. The phenomenon of dynamic amplification was not observed in any case, since the frequencies of the load were quite lower from the fundamental frequency of the structure.
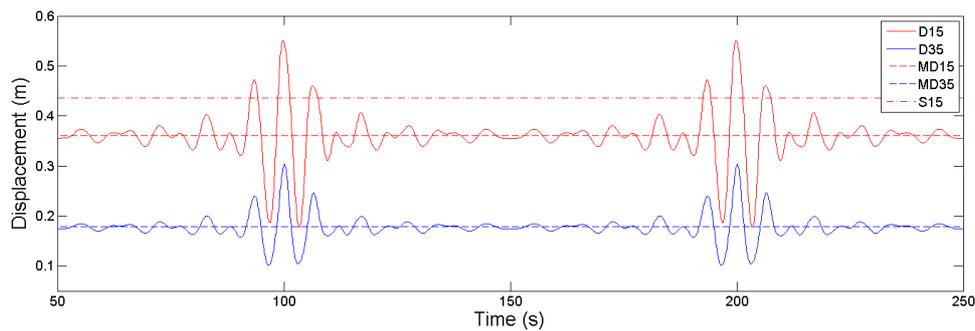
Figure 8: Dynamic, dynamic mean and static displacement, at the top of the mast, for 15 kN and 35 kN of pretension.

Figure 8 shows the extreme cases of pretension (15 kN and 35 kN), where *D* indicates dynamic displacement, *MD* indicates the mean of dynamic displacement and *S* the value of displacement with static load. The number *15* or *35* indicates the pretension (kN). The principal difference between the two cases lies in the mean value and the amplitude of the motion, both are duplicated from pretensions 35 kN to 15 kN. Figure 9 illustrates the evolution of the absolute maximum, the absolute minimun and the mean displacements for dynamic load, and the value for static load, for each case of pretension. The dynamic mean match the static value of displacement for 30 kN and 35kN, while is always higher in the other cases. It can be seen that the maximum amplitude of motion reduces smoothly when the pretension increases.
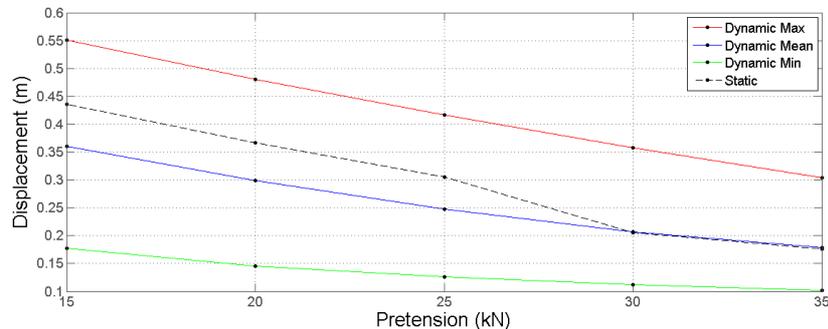


Figure 9: Dynamic absolute maximum, minimun, mean and static values of displacement, at the top of the mast, for all pretension studied.

Figure 10 shows the maximum dynamic (MaxD) deformation and the static (S) deformation of the whole mast in the load direction. It shows how the increase of pretension stiffens the structure, limiting its movements and preventing the rotation at the top.

Finally a FFT study for the displacements at the top was carried out. Figure 11 shows the FFT of the load and pretensions of 15 kN and 35 kN. The FFT analysis reveals that the mast, in both cases (and the intermediate -non shown herein- too), exhibit peaks of frequency according with the load frequencies. Moreover, it can also can be seen that the dynamic variation is higher for lower prestress levels.

## 2.2.2 Dynamic tension on guys

The evolution of the guys dynamic tension, for each pretension, was reported for the $a1$, $a4$, $b1$ and $b4$ guys, were $a$ or $b$ denotes the direction of the guy and the number 1 or 4 indicates
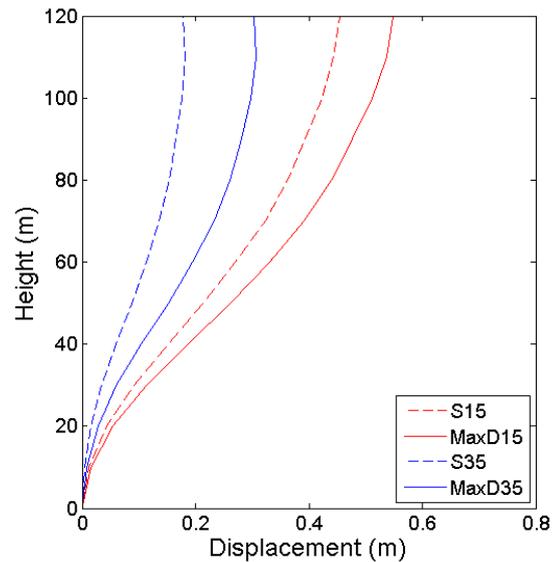
Figure 10: Dynamic and static displacements of the mast corresponding to peak loads
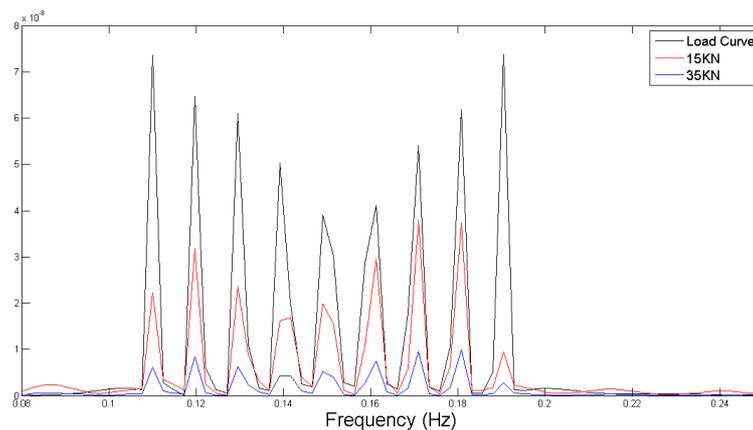


Figure 11: FFT analysis of the displacements at the top

the guy level. For further illustration, see Fig. 2.

Figure 12 shows the dynamic tension vs pretension case. The guys in the $a$ direcion lose tension due the deformation of the mast. For a lineal increment of the pretension, the maximum, the minimum and the mean dynamical responses result in a nonlinear increase. The difference between the maximum and the minimun values also increases. The guys in the $b$ direction show higher values than the initial prestress. The response of the studied parameters follows a linear growth and the difference between the maximum and the minimun become smaller: the dynamic behaviour tends to the mean.

Figure 13 illustrate the dynamic tension on guys as a percentage of initial pretension vs initial pretension. The cables in the $a$ direction have a mean value of dynamic tension between 65% and 30%, and the difference between the maximum and the minimun remains within the range of 20% of initial prestress. It also can be seen that the trend to lose tension (slope of the curves) becomes smaller or even reverts for higher pretension. In the cables in the $b$ direction the mean dynamic tension decreases from about 180% to 100%. The difference between the maximum
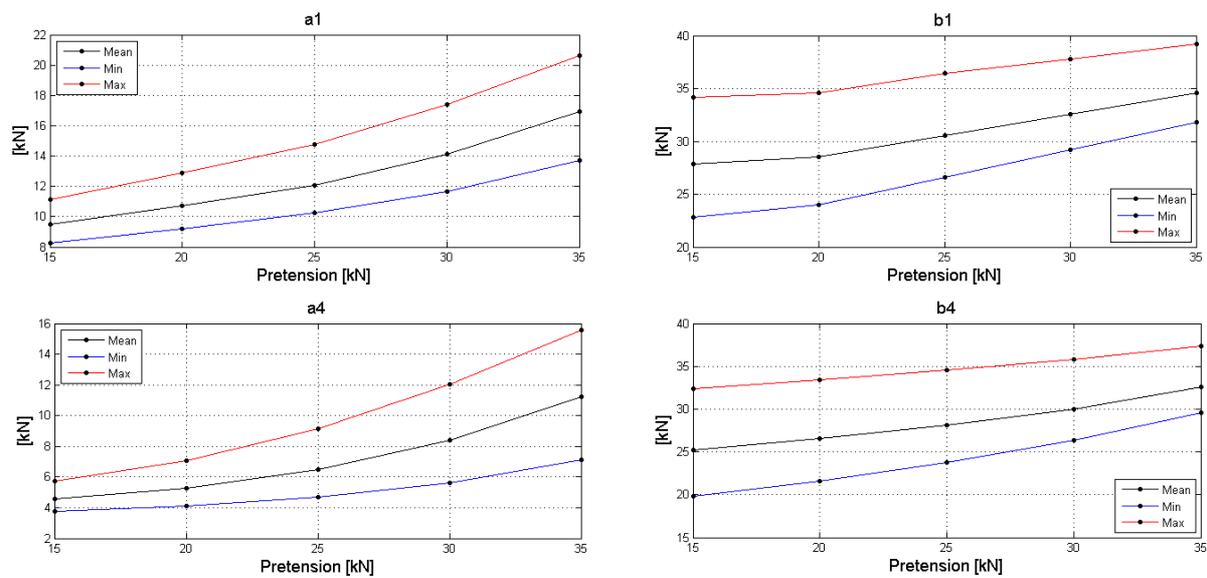
Figure 12: Maximum, mean and minimum dynamic tension on guys
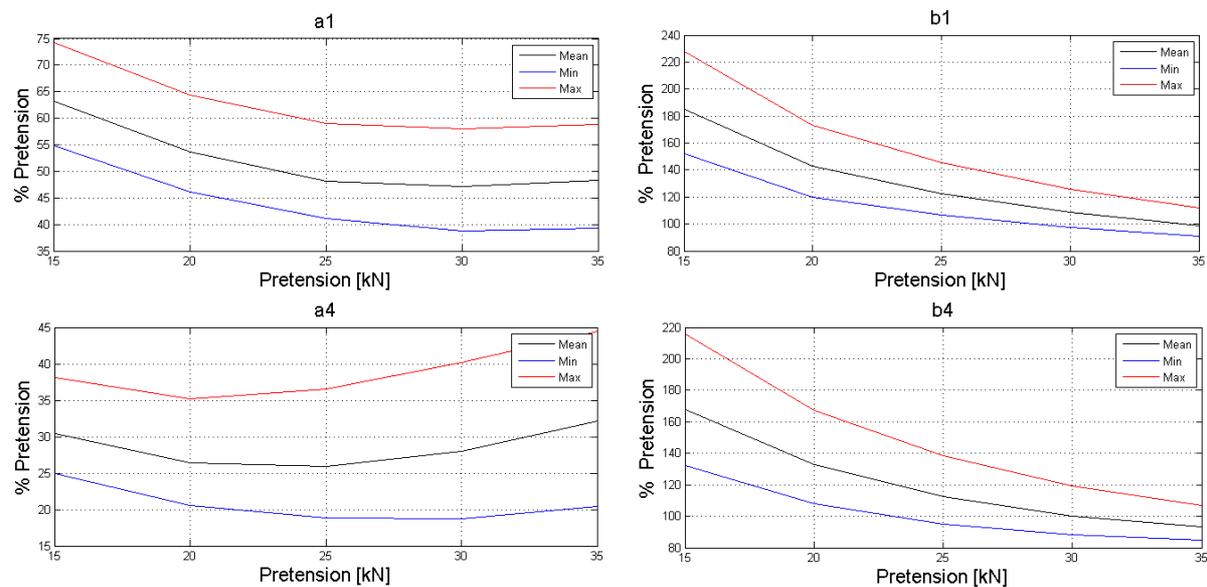
and the minimun also decreases from 80% to 30%.



Figure 13: Maximum, mean and minimum dynamic tension on guys as percentage of intial pretension

## 3  SEISMIC LOADED MODEL

Here, a study of a guyed mast under seismic load is briefly reported. For more details see Guzmán et al. (2010). The mast is a beam-column, made of 12 6-DOF beam elements and it is fixed at base. Each guy is modeled using 5 3-DOF pre-stressed cable elements, considering the lack of compressive resistance. To find the dynamic non-linear response the finite element software SAP2000 (2007), that allows consideration of large displacements for the cables and geometric non linearity for the mast, was used.

### 3.1 Seismic Load

The guyed masts dynamic response was also analyzed using four seismic load records:

- San Juan, November 23th, 1977, INPRES, EW, magnitude of 7.5 on the Richter scale, duration 55 s, PGA (peak ground acceleration) 0.193 g.

- Mendoza, January 26th, 1985, Las Heras City Hall, NS, magnitude of 5.9, 3 s of duration, PGA 0.408 g.

- Valparaiso, March 3th, 1985, Lloleo, horizontal component 10, magnitude 7.8, duration 70 s, PGA 0.669 g.

- Northridge, January 17th, 1994, 24514 Sylmar - Olive View Med FF, horizontal component Syl360, magnitude of 6.7, duration 20 s, PGA 0.843 g.

  Figure 14 shows the time history of the earthquake records. Both San Juan and Valparaiso accelerograms are vibratory type earthquakes, while Mendoza and Northridge have the characteristics of impulsive earthquakes.
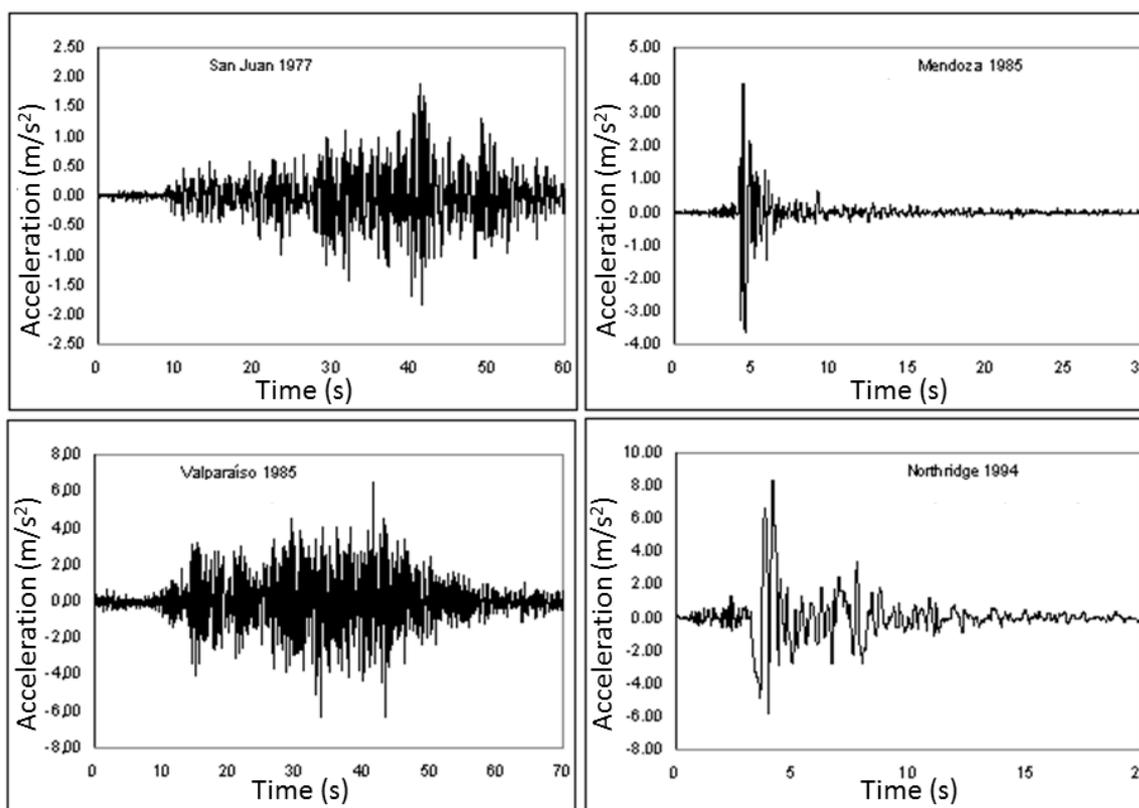


Figure 14: Analized earthquake records

## 3.2   Response of the system with the different pretensions

Figure 15 shows the effect of pretension on the maximum value of the dynamical displacements at the top of the mast. When the accelerograms of Mendoza (M) and Northridge (N) were applied, the maximum displacements tend to sligthly decrease with the pretension. In the case of the accelerograms of San Juan and Valparaiso no trend is apparent.
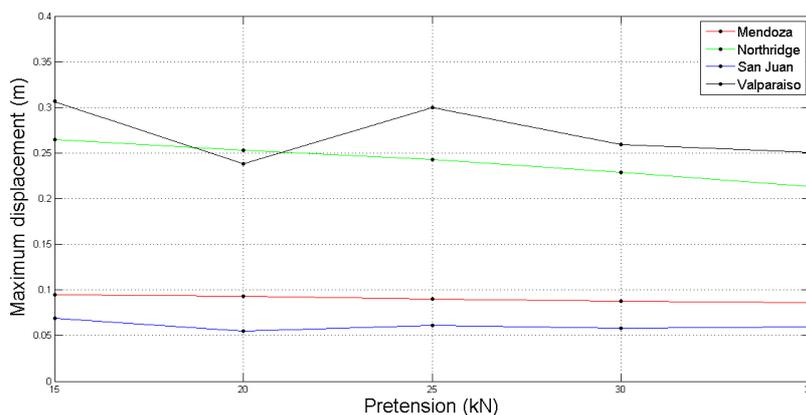


Figure 15: Pretension effect on maximum displacements

Figure 16 depict the effects of different pretension on maximum and minimun tension on the $2a$ and $2b$ guys, respectively.
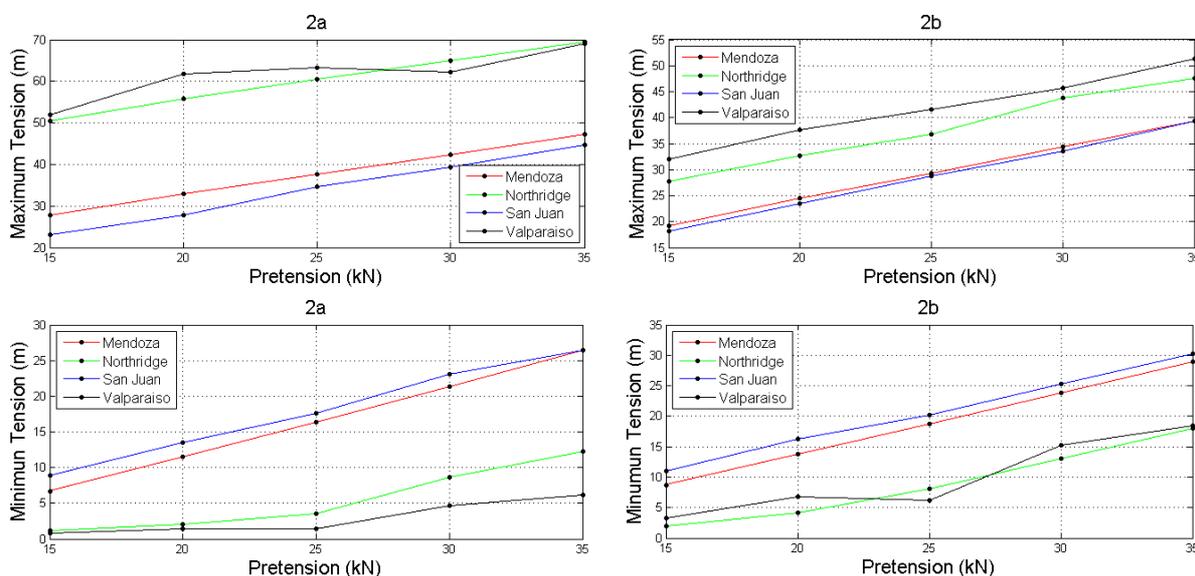


Figure 16: Pretension effect on maximum and minimun dinamic tensions on the $2a$ and $2b$ guys

The maximum dynamic tension grows linearly with pretension. This behaviour does not appear to be dependent on the characteristics of the accelerogram. In all cases the $2a$ guy was the most taut, as expected. It observed that the increase of the dynamic tension was higher for lower pretensions, reaching the 240% on the Northridge and Valparaiso cases.

The dynamic decrease of pretension was more noticeable on the $2a$ guy and was higher for lower pretensions in all cases. Finally, although loss of tension was not presented in any of the cases analyzed, when the system was subject to the accelerograms of Valparaiso and Northridge, very low values were observed.

## 4 CONCLUSIONS

In this work a dynamic, non-linear finite element analysis of a guyed mast was carried out, in order to analize the influence of pretension of the guys on the dinamic response of the system under wind load and different kinds of earthquake actions. The wind study was described in detail and the earthquake study and results are treated briefly.

In the wind load case, the structure moves following the same frequencies of the load, as the FFT study shows. Anyway no resonance phenomena is observed, due the fundamental frequency of the mast is higher than the load frequencies.

For the earthquake model four accelerograms were used. The Valparaiso and Northridge earthquakes have a strong influence on the dynamic response of the system, when evaluated under different levels of pretension. Since the maximum response was obtained for both a vibratory (Valparaiso) and a impulsive (Northridge) earthquake, it is not possible to draw conclusions that depend on the earthquake type.

In the analysis of the displacements on the top of the mast, the first notorious difference is that, in the wind case, a significant decrease is present when pretension increases; in the earthquake case, this trend is smaller or even inexistent.

In the case of guys tension, the behaviour was completely different between wind and eartquake actions. The first discrepancy is that, in the wind case the most stressed guy is the $b$ direction meanwhile under earthquake load is the $a$ direction. Secondary the evolution undher wind loads of dynamic tension with the increment of pretension result in a curve increasing from the lower values and other times with a linear fashion. The largest increment was around 100% of the initial pretension. Unlikely, the linear variation is frequently presente for the earthquake case, although the tend is the same. The guys are overstressed in as much as 240% of intial pretension.

The standards recommend a pretension of the cables in the range of 8% to 15%. However, from the seismic analysis it is apparent that a pretension value in that range does not assures a proper behaviour, since the response is more influenced by the earthquake characteristics than the pretension level. In the wind case, the tower becomes stiffer as the pretension is increased. The recommended range of pretension appears to be convenient for the studied guyed tower and wind load.

## REFERENCES

ALGOR-Inc. *ALGOR V23.01. Professional MES*, 2009.

Castro H.G., De Bortoli M.E., Wittwer A.R., and Marighetti J.O. Simulación numérica del campo de velocidades del viento atmosférico utilizando el método de representación espectral. In *ENIEF 2007*. 2007.

CIRSOC-INTI. *Reglamento CIRSOC 102. Acción del Viento sobre las Construcciones (en trámite de aprobación).* INTI, Buenos Aires, Argentina, 2005.

de Oliveira M.I., da Silva J.G., da S. Vellasco P.C., de Andrade S.A., and de Lima L.R. Structural analysis of guyed steel telecommunication towers for radio antennas. *J. of the Braz. Soc. of Mech. Sci. and Eng.*, 29:185–195, 2007.

Dyrbye C. and Hansen S. *Wind Loads On Structures*. John Willey and Sons, West Sussex, England, 1 edition, 1994.

Guzmán M., Rosales M., and Filipich C. Respuesta sísmica de un mástil arriostrado. In *Jornadas Sudamericanas de Ingeniería Estuctural*. 2010.

Horr A.M. Nonlinear spectral dynamic analysis of guyed towers: Part i: Theory. *Canadian Journal of Civil Engineering*, 31:1051–1060, 2004.

Kahla N.B. Equivalent beam-column analysis of guyed towers. *Computers and Structures*, 55(4):631–645, 1993.

Kahla N.B. Response of a guyed tower to a guy rupture under no wind pressure. *Engineering Structures*, 22:699–706, 2000.

Kewaisy T.H. *Nonlinear Dynamic Interaction between Cables and Mast of Guyed-Tower Systems Subjected To Wind-Induced Forces*. Ph.D. thesis, Texas Tech University, 2001.

Lu L., Qu W., and Li M. Simulation of wind velocity and calculation of wind load for guyed masts. *Wuhan Ligong Daxue Xuebao (Jiaotong Kexue Yu Gongcheng Ban)/Journal of Wuhan University of Technology (Transportation Science and Engineering)*, 34:1057–1060, 2010.

Matuszkiewicz M. Calculation of guyed masts in accordance with en 1993-3-1 standard taking into account mast shaft geometrical imperfections. *Engineering Structures*, 33:2044–2048, 2011.

Meshmesha H., Sennah K., and Kennedy J.B. Simple method for static and dynamic analyses of guyed towers. *Structural Engineering and Mechanics*, 23(6):635–649, 2006.

Preidikman S., Massa J., and Roccia B. Análisis dinámico de mástiles arriostrados. *Rev. Int. de Desastres Naturales, Accidentes e Infraestructura Civil*, 6(1):85–102, 2006.

Punde Y.D.S. Simple model for dynamic analysis of cable supported structures. *Engineering Structures*, 23:271–279, 2001.

SAP2000. *SAP2000, CSi*, 2007.

Shi H. *Nonlinear Finite Element Modeling and Characterization of Guyed Towers Under Severe Loading*. Ph.D. thesis, University of Missouri, Columbia, 2007.

Wahba M., Madugula M., and Monforton G. Evaluation of non-linear analysis of guyed antenna towers. *Computers and Structures*, 68:207–212, 1998.

Wahba M.M.Y. and Monforton G. Dynamic response of guyed masts. *Engineering Structures*, 20(12):1097–1101, 1998.