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Dynamic Response of a Guyed Mast under Seismic Loadings

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Abstract: ***The use of guyed masts to support communication antennas is much extended. In the design stages or reinforcement of these systems, uncertainties related with the structural dynamic behavior are unavoidable, such as the variability's of the guy pretension, the equivalent damping and the bending stiffness of the mast. The aim of the present work is to evaluate the parametric sensitivity of a guyed mast under seismic loading. The problem is solved in a finite element environment under the action of six seismic records of different earthquakes. The sensitivity of the resulting base shear and the horizontal displacement at the top to the variation of the guy pretension, the equivalent damping and the bending stiffness, is assessed. The changes in these parameters values modify significantly the dynamic structural response.***

Keywords: guyed mast, seismic response, earthquakes, parametric study.

1. Introduction

Guyed masts (Figure 1) are distinguished for being tall, slender and mainly, flexible which makes them especially sensitive to dynamic excitation. These systems consist of two main elements, a mast and the guys at different heights, resulting in an inherent non-linear structure even during the usual performance, mostly due to the guys behavior [1]-[2]-[3].

Figure 1: Typical guyed mast

This structural typology is frequently used to support instantaneous voice transmission, mobile phone, radio and TV industries and emergency devices that use radio and phone signals. Despite the great potential of adverse impact, dynamic loads are usually simplified, however, their study becomes relevant in various situations. Saudi [4] reports the structural assessment of a guyed mast using dynamic quantities. Relevant researches have been carried out considering the wind action [5]-[6]. On the other hand, the initial geometrical imperfections of the mast should not be neglected in the design [7]. As for the seismic loads, are not usually considered as dominant actions at the design stage, though published studies have shown that the demand on the system posed by some seismic records can result crucial to the operative continuity of the communications [3]-[8]-[9]. An example of this was what happened in the earthquake in Chile in 2010, where in the regions of Maule and Biobio, one of the most affected, many antennas changed position and lost communication with their peers [10].

It is observed that the study of the seismic action on these structural systems has not been extensive. Few researchers have tackled this subject, e.g. Amiri and McClure [11]; Amiri [3]; Hensley and Plaut [12]; Faridafshin and McClure [13]; Grey et al. [14]. More recent work has addressed the behavior of guyed masts with faults under seismic excitations [15]-[16]. Some non-official reports of damage during earthquakes can be found in Madugula [17]. The aim of the present work is to assess the sensitivity of a particular guyed mast under a series of seismic records. A detailed study is carried out using a finite element model and using a range of values for the three proposed parameters subjected to various seismic records in order to analyses the effect of the changes of both structural and load features in the dynamic response.

2. Guyed mast

The system consists of a 120 m mast with three guys (cables) per each of the four levels, arranged in a 120° layout (Figure 2). The mast is hinged at the base and modeled as an equivalent beam-column (a number of 16 beam type elements are used in the Finite Element Method discretization). The behavior of this typology of masts was opportunely studied by the author [18]. The guys are modeled as pretensioned cables with five finite elements which include the extensibility and account only for the tensile capacity, i.e. the cables do not support compressive load. The software SAP2000 [19] was employed to solve the problem. It allows for the geometric nonlinearities of both the mast and guys as well as the second order effect in the mast due to axial loads.

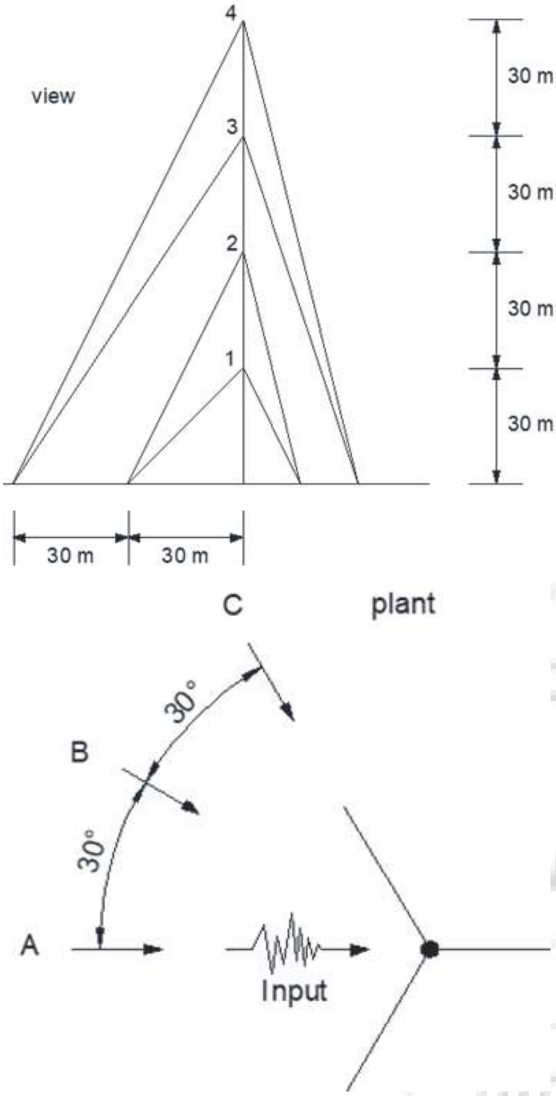


Figure 2: Geometry of the guyed mast analyzed

3. Seismic Records

The main data of the records employed in the present study are depicted in Table 1.

Table 1: Data of the seismic record used in the present study. *[20]-**[21]

Records	*Data*	*M [Richter]*	*PGA [g]*
*San Juan (SJ)	23-Nov-1977	7.4	0.193
*Mendoza (M)	26-Jan-1985	5.9	0.408
**Tabas (T)	16-Sep-1978	7.8	0.850
**Valparaiso (V)	03-Mar-1985	7.8	0.669
**Northridge (N)	17-Jan-1994	6.7	0.843
**Kobe (K)	17-Jan-1995	7.2	0.821

3.1 Local records (Argentina)

The E-W component of the San Juan (SJ) earthquake (Figure 3) was recorded at the station located in the National Institute of Seismic Prevention headquarters (INPRES, acronym in Spanish). Regarding the Mendoza (M) earthquake, the N-S component was selected (Figure 3) from the records of the Las Heras municipality station (INPRES, 1986). The data of the SJ and M records were obtained from Frau [20]. These particular earthquakes were selected since they represent the seismicity of the Argentinian central-west region, location of the largest earthquakes in Argentina.

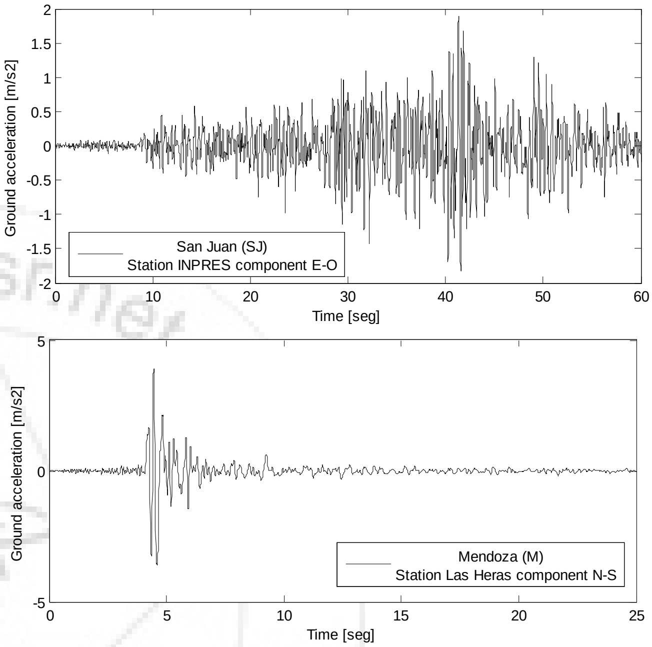


Figure 3: Seismic local records considered

3.2 International records

The TR horizontal component of the Tabas earthquake (T) was recorded at the Tabas station, Iran. Also, the Valparaiso (V) earthquake registered at the Llolleo station, Chile is considered through the horizontal component 10. In the Northridge (N), USA earthquake, the Syl360 earthquake component recorded at the 24514 Sylmar - Olive View Med FF station is considered in the study. The 0 horizontal component of the Kobe (K), Japan of the 0 KJMA station is. All these accelerograms (Figure 4) were obtained from PEER [21].

The Valparaiso record was selected due to its world relevant magnitude and furthermore, since the effects of these interplate earthquakes have a strong influence in the central-west region of Argentina. Meanwhile, the Northridge, Tabas and Kobe records were included in the present study due to their important magnitudes and impulsive characteristic. The latter allows to assess the behavior of the structure under this near-fault earthquake (interplate or cortical ground motion). It is known that they can lead to different structural response in comparison to the far-fault motions. Given the selected earthquakes, it is apparent that the study encompasses different seismic features such as durations and frequency content.

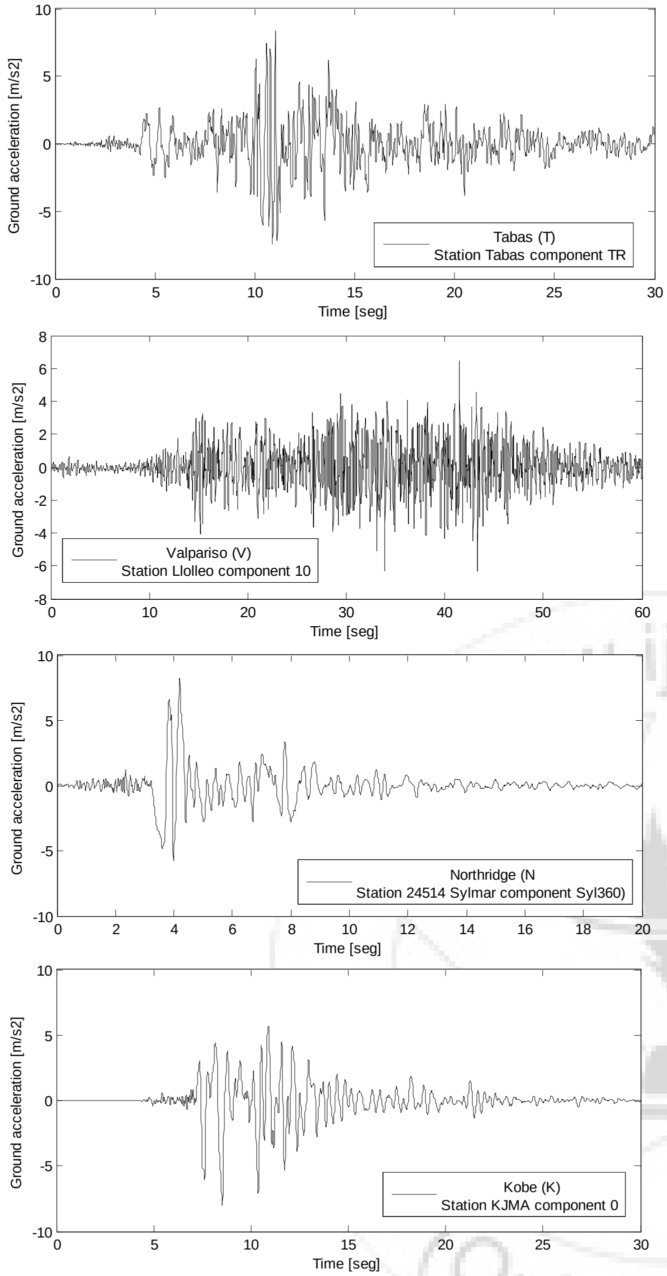


Figure 4: Seismic international records considered

4. Design parameters

At the design stage or in the need of retrofitting, some uncertainties related to the dynamic modeling arise. They can be the level of initial pretension of the guys, the equivalent Rayleigh damping coefficients and the second area moment of the cross section of the mast (proportional to the bending stiffness).

Thus, and in order to evaluate the influence of these variations, a sensitivity parameter study is carried out through numerical simulations considering different possible combinations of the selected parameters.

4.1 Guys initial pretension (IP)

The initial pretension represents the tensile force given to the guys that contributes to the structural system stiffness. Usually this pretension is represented not by a force but a stress (i.e., force over cross section area). The design values suggested by the codes are expressed in a range of percentages of the ultimate strength, e.g., from 8 to 15 % in the Canadian Standard [22] or from 7 to 15 % in the ANSI/TIA Standard [23].

The value of this pretension should remain during the service life of the structure. However, its magnitude can be altered by different reasons and the frequent monitoring becomes necessary. Since many of these structures are installed at far locations and the necessary devices are not available in some cases, the design guy pretension exhibits variabilities, even beyond the recommended ranges. Lower or higher values of the cable pretension can modify the stability of the whole structure. In the present study, in order to cover the standards suggested range and taking into account the guys cross section area and the material, the following pretension forces are considered: 15.0, 20.0, 25.0, 27.5, 30.0, 32.5 and 35.0 kN (taking a range of 5-12 % of the ultimate strength).

4.2 Equivalent structural damping (D)

The well-known Rayleigh model is based on a damping matrix that is proportional to the mass and stiffness matrices. It leads to a decoupling of the damping matrix (Thomson, 1982) [24]. An appropriate value of the damping coefficient is necessary to account for energy dissipation that reduces the vibrations due to earthquakes or wind.

Experimental measurements of a guyed lattice mast [25] led to an equivalent damping ranging between 1 and 3 % of the critical value, with an average of 1.6 %. On the other hand, the IASS standard [26] recommends a value of 3 % for steel structures with bolted joints, the CIRSOC code [27] adopts 2 % while the ANSI/TIA standard sets a value of 5 % for a temporal seismic analysis. The literatures reports values among 1 and 5 % of the critical value [3]-[12]. In the present study, and taking into account the above data, three cases of damping are considered, i.e. 1, 2 and 3 % of the critical damping. The proportional coefficients of the mass and stiffness matrices for each damping level are 0.118 and 0.00070, 0.237 and 0.00144 and, 0.356 and 0.00209, respectively.

4.3 Mast bending stiffness (MS)

The second area moment I of the mast cross section (proportional to the bending stiffness EI) is fully determined at the design stage. However, the retrofitting of the structure, e.g. due to new antennas installation, can lead to the reinforcement of the mast legs and a change of the bending stiffness. To account for this variability, three values of I are considered: 1.80 10-3, 2.25 10-3 y 2.70 10-3 m^4.

4.4 Combination of the design parameters

The different cases arise from the possible combination of the design parameters that were considered variable (7 pretension, 3 damping and 3 mast second moment of area values) giving place to a total of 63 model cases (see Table 2). A notation is introduced to identify the cases: IP-D-MS. For instance, 15.0-1-1.80 denotes an initial pretension of 15 kN, a relative damping of 1 % and a mast second moment of area of 1.80 x 10-3 m^4.

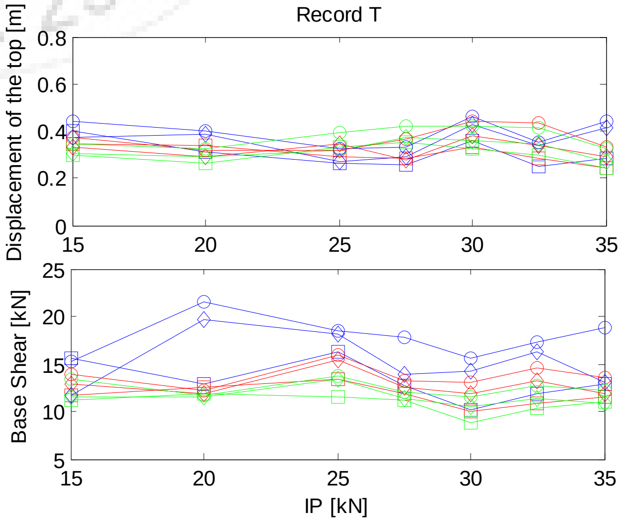
Table 2: Models considered in the analysis

Model	*Pretension IP [kN]*	*Damping D [%]*	*Moment MS [m^4]*
15.0-1-1.80	15.0	1	0.00180
15.0-1-2.25	15.0	1	0.00225
15.0-1-2.70	15.0	1	0.00270
15.0-2-1.80	15.0	2	0.00180
15.0-2-2.25	15.0	2	0.00225
15.0-2-2.70	15.0	2	0.00270
15.0-3-1.80	15.0	3	0.00180
15.0-3-2.25	15.0	3	0.00225
15.0-3-2.70	15.0	3	0.00270
20.0-1-1.80	20.0	1	0.00180
20.0-1-2.25	20.0	1	0.00225
20.0-1-2.70	20.0	1	0.00270
20.0-2-1.80	20.0	2	0.00180
20.0-2-2.25	20.0	2	0.00225
20.0-2-2.70	20.0	2	0.00270
20.0-3-1.80	20.0	3	0.00180
20.0-3-2.25	20.0	3	0.00225
20.0-3-2.70	20.0	3	0.00270
25.0-1-1.80	25.0	1	0.00180
25.0-1-2.25	25.0	1	0.00225
25.0-1-2.70	25.0	1	0.00270
*25.0-2-1.80	25.0	2	0.00180
25.0-2-2.25	25.0	2	0.00225
25.0-2-2.70	25.0	2	0.00270
25.0-3-1.80	25.0	3	0.00180
25.0-3-2.25	25.0	3	0.00225
25.0-3-2.70	25.0	3	0.00270
27.5-1-1.80	27.5	1	0.00180
27.5-1-2.25	27.5	1	0.00225
27.5-1-2.70	27.5	1	0.00270
27.5-2-1.80	27.5	2	0.00180
27.5-2-2.25	27.5	2	0.00225
27.5-2-2.70	27.5	2	0.00270
27.5-3-1.80	27.5	3	0.00180
27.5-3-2.25	27.5	3	0.00225
27.5-3-2.70	27.5	3	0.00270
30.0-1-1.80	30.0	1	0.00180
30.0-1-2.25	30.0	1	0.00225
30.0-1-2.70	30.0	1	0.00270
30.0-2-1.80	30.0	2	0.00180
30.0-2-2.25	30.0	2	0.00225
30.0-2-2.70	30.0	2	0.00270
30.0-3-1.80	30.0	3	0.00180
30.0-3-2.25	30.0	3	0.00225
30.0-3-2.70	30.0	3	0.00270
32.5-1-1.80	32.5	1	0.00180
32.5-1-2.25	32.5	1	0.00225
32.5-1-2.70	32.5	1	0.00270
32.5-2-1.80	32.5	2	0.00180
32.5-2-2.25	32.5	2	0.00225
32.5-2-2.70	32.5	2	0.00270
32.5-3-1.80	32.5	3	0.00180
32.5-3-2.25	32.5	3	0.00225
32.5-3-2.70	32.5	3	0.00270
35.0-1-1.80	35.0	1	0.00180
35.0-1-2.25	35.0	1	0.00225
35.0-1-2.70	35.0	1	0.00270
35.0-2-1.80	35.0	2	0.00180
35.0-2-2.25	35.0	2	0.00225
35.0-2-2.70	35.0	2	0.00270
35.0-3-1.80	35.0	3	0.00180
35.0-3-2.25	35.0	3	0.00225
35.0-3-2.70	35.0	3	0.00270

*standard case

5. Seismic Response

The structural response is analyzed in the direction named A shown in Figure 2 which is coincident with the soil motion imposed by the input records. The seismic response of the structure is analyzed through the displacements at the top of the mast (relative to the ground motion) and the base shear. Figure 6 show the outcomes for all the models and records. It is observed that with almost all the records and with the higher values of D and MS (3 % and 2.70 10-3 m4, respectively), the guys IP has little influence on the response. The top displacements using the Tabas, Valparaíso and Kobe records are of 0.4 m. A maximum value of 0.62 m was attained with the model 15.0-1-1.80 with the Kobe record. The variation of the maximum values of the base shear follows the changes in the peak ground acceleration (PGA). That is, the larger the PGA, the larger the base shear. The weight of the mast is around 71.8 kN while the maximum base shear in all cases is on the order of 4 to 30 % of this weight. The largest base shear resulted in 23.0 kN (model 15.0-1-1.60, Kobe record). It is worth of mention that the structure response is strongly modified as IP and/or MS change, more noticeable with the models of smaller values of D and MS.



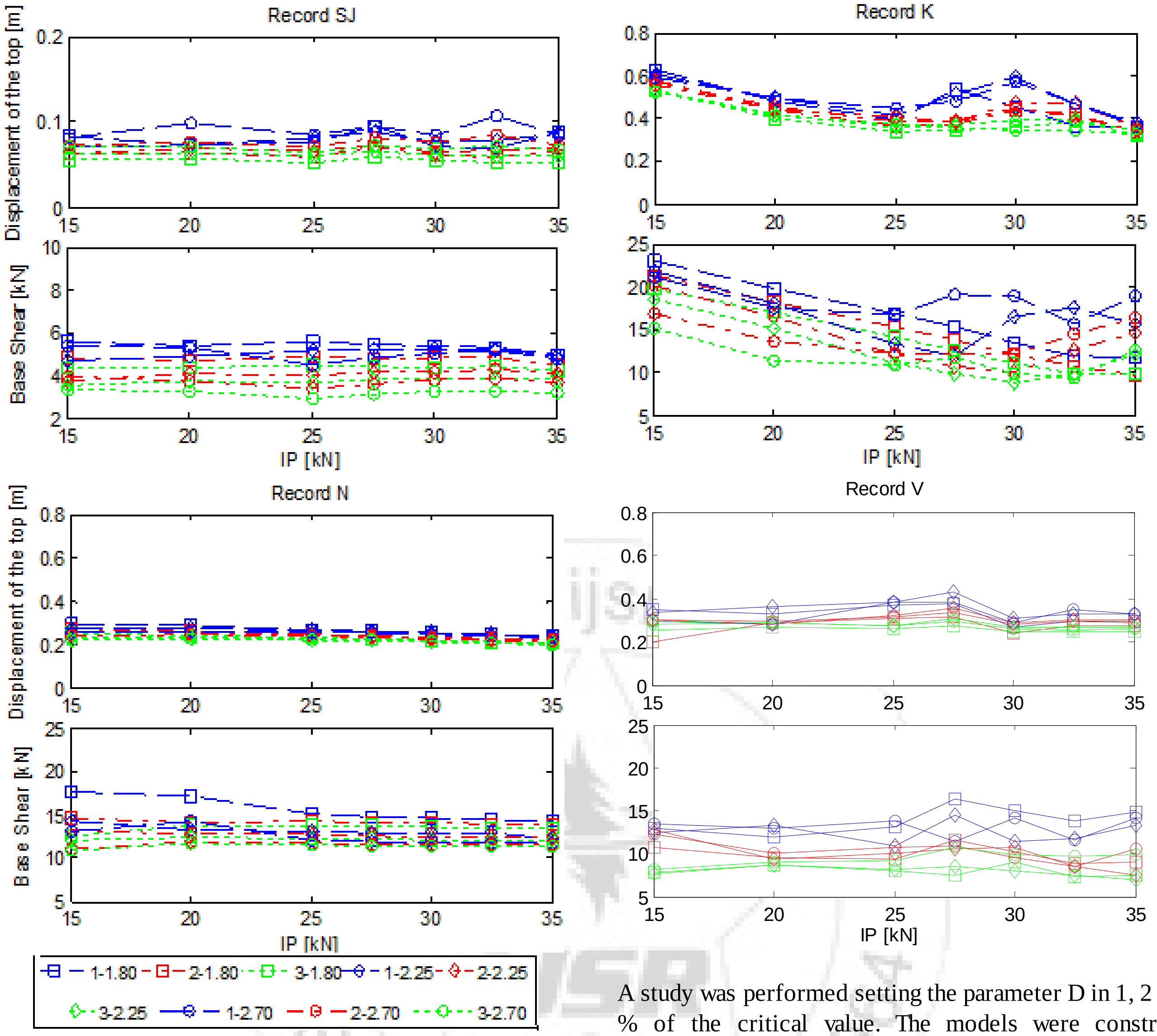
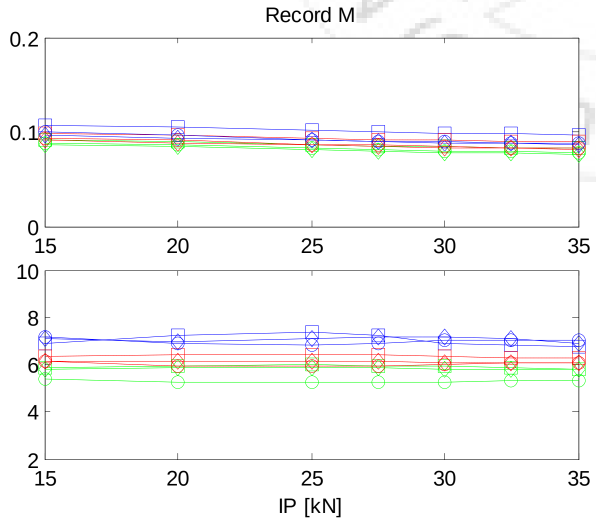


Figure 6: Absolute maximum responses (top displacement and base shear) for the records



A study was performed setting the parameter D in 1, 2 and 3 % of the critical value. The models were constructed varying IP and MS and under all the seismic records. Figures 7 to 9 depicts the results for each value of D, respectively. It shows how the seismic response is strongly dependent on the particular characteristics of each record, and how, for some records, such responses vary when the IP is modified.

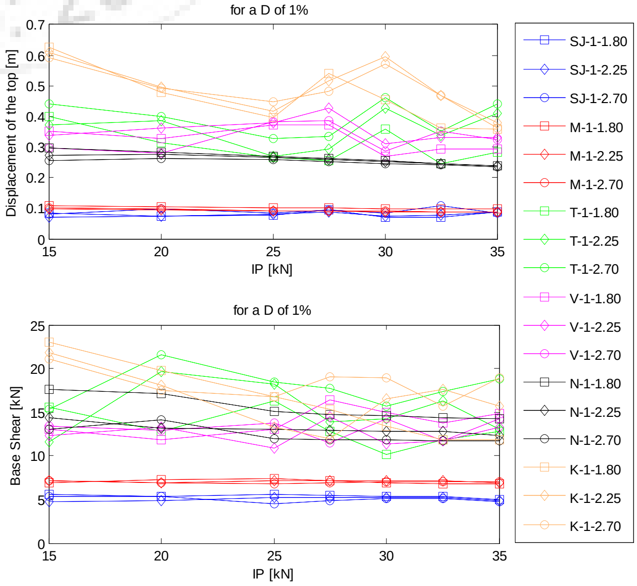


Figure 7: Absolute maximum responses for a D of 1 %

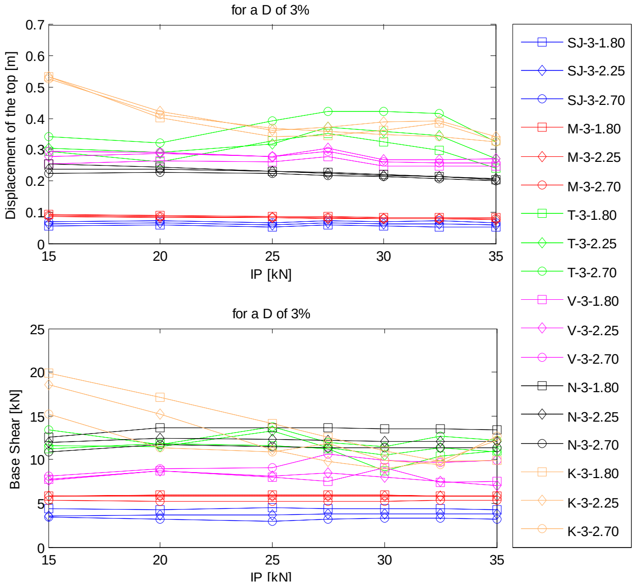


Figure 8: Absolute maximum responses for a D of 2 %

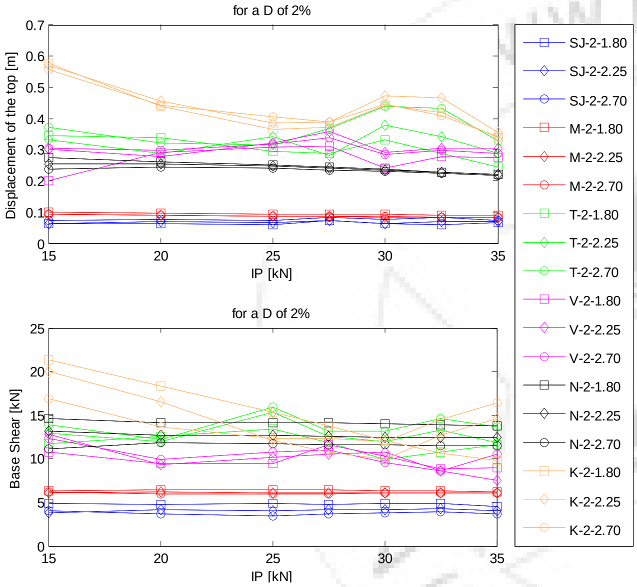


Figure 9: Absolute maximum responses for a D of 3 %

The time histories of the displacements and base shear found for the cases 15.0-1-1.80, 27.5-1-1.80 and 35.0-1-1.80 under the Kobe record are shown in Figures 10 and 11. Also, the maximum absolute values are indicated with circles.

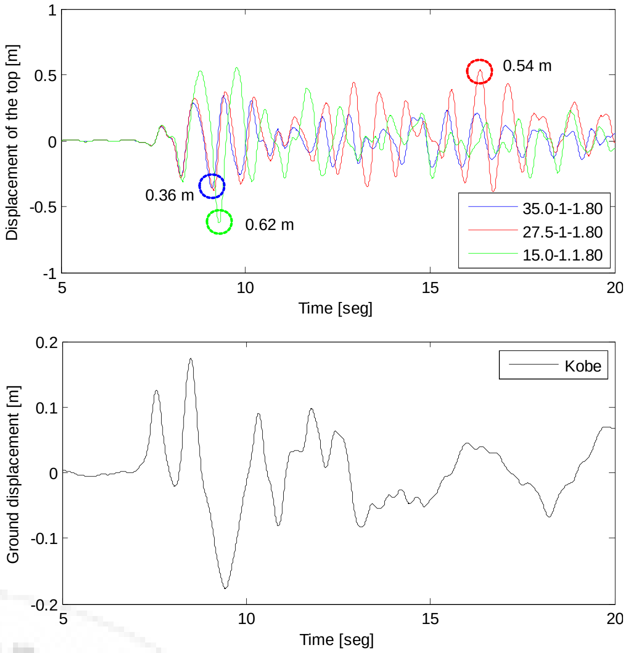


Figure 10: Time history of the displacement the top using three different IP values under Kobe record

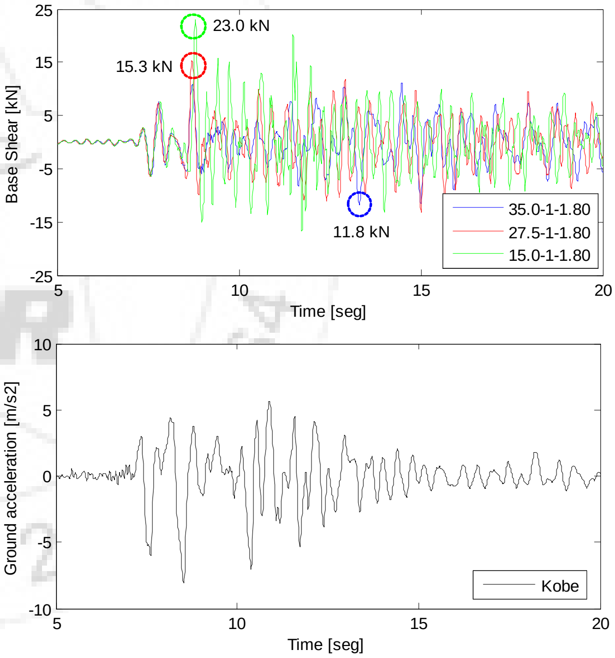


Figure 11: Time history of the base shear using three different IP values under Kobe record

Additionally, a Fast Fourier Transform (FFT) was applied to the time history of the maximum displacement at the top of the mast. It was found that a peak within a range of 1 to 1.5 Hz is always present. Figure 12 depicts the FFT of the earthquake accelerograms. The two vertical full lines indicate this frequency range while the dotted line denotes the fundamental frequency of the mast obtained with the reference model (25-2-1.80).

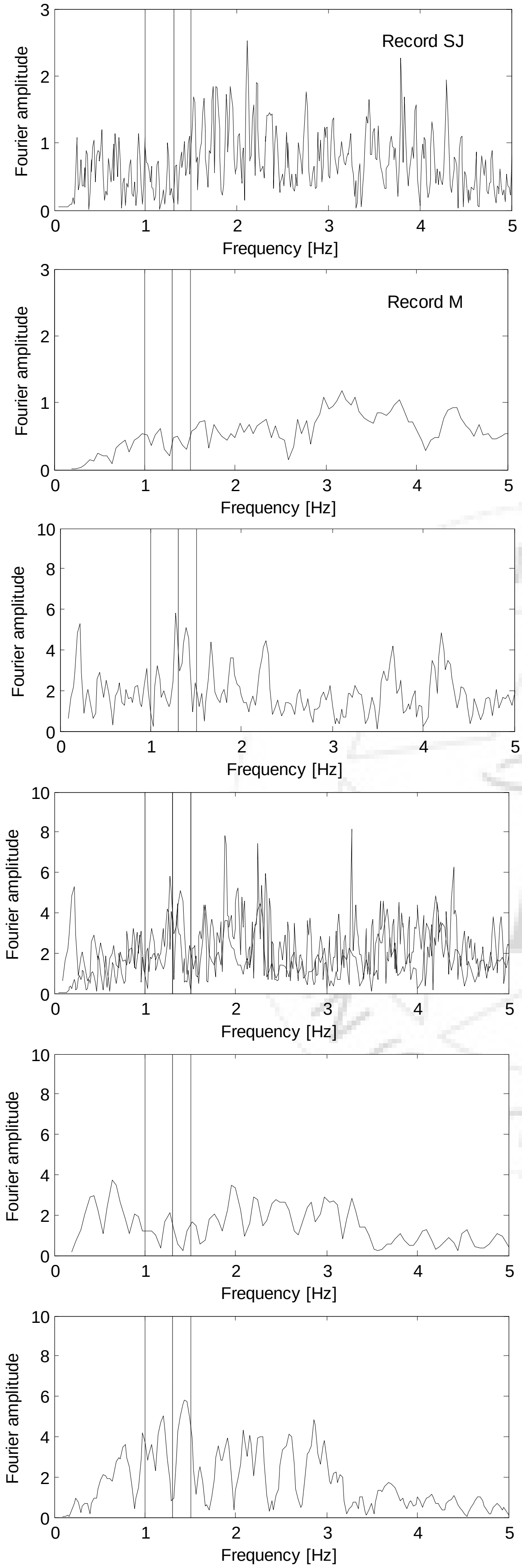


Figure 12: Fast Fourier Transform of the maximum top displacement

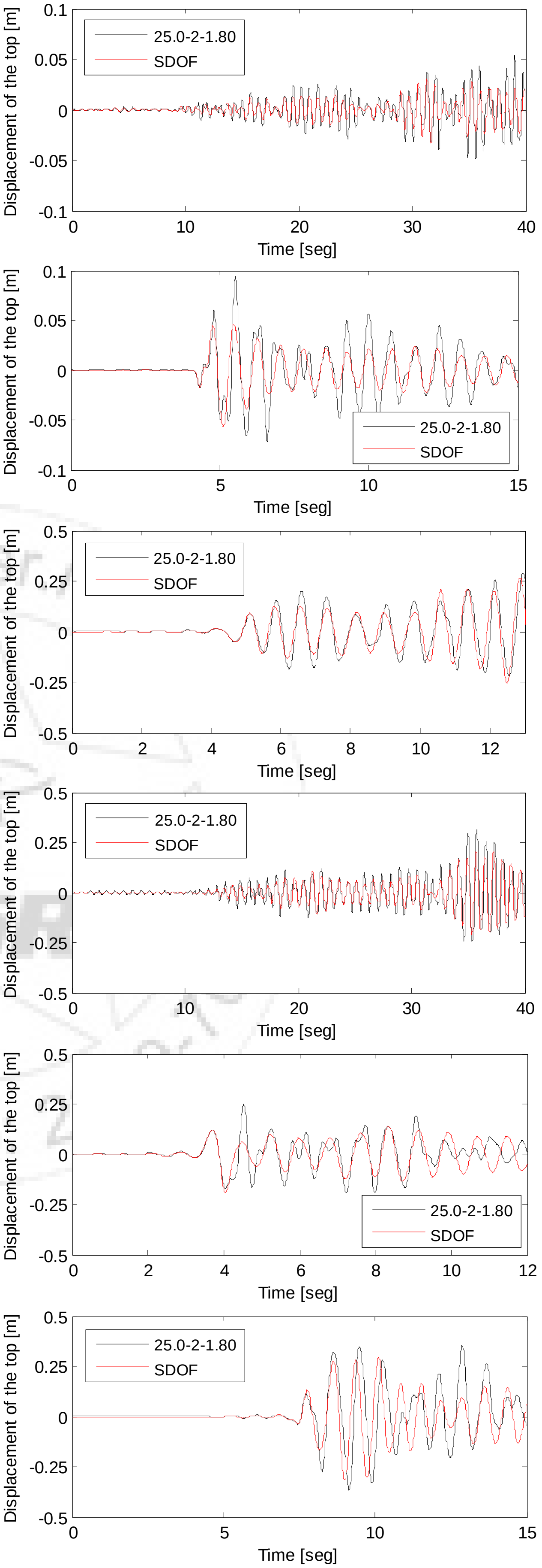


Figure 13: Comparison between responses of top displacements using the SDOF and full models

In the cases in which the accelerograms present a higher content of frequencies in the mentioned range, the top displacements increase, though no define trend can be established. On the contrary, a lower content of energy in that range leads to smaller displacements with a clear pattern.

Since the results indicate that there is a considerable frequency content close to the fundamental frequency of the mast, an equivalent linear SDOF model was constructed for the standard model (25-2-1.80). It was found that this strongly simplified model can reproduce rather closely the top displacements frequency content, although the representation of the amplitudes is not adequate. However, the SDOF model could serve as a first, rough estimation of this complex problem. Figure 13 shows a comparison of this results.

6. Conclusions

A detailed nonlinear dynamic study of a guyed mast subjected to seismic action was presented. A parametric analysis was carried out considering six records of different earthquakes and variations in the design parameters guy initial pretension, damping and mast stiffness. Their combination gives place to 63 cases under the six seismic records. The continuous structure was discretized with finite elements.

- First, the finite element model developed using SAP2000 with verified with respect to the results of natural frequencies obtained by Hensley and Plaut using ABAQUS. The evaluation established an excellent performance of the software used in this work.
- In the parametric study, it is observed that as the structural damping is increased from 1 to 3 %, almost all the dynamic responses evaluated in the 63 cases, decrease between 5 and 40 %, depending on the considered guys IP level, MS and seismic record.
- In most of the cases and when the adopted values of D and MS are the largest, the system responses are little affected by the different values of IP.
- Relative significant demands are observed in the response in relation with the sensitivity levels required to attain the desired signal transmission quality. The maximum values were 0.62 m for the top displacement (0.52 % of mast height) and 23.0 kN for the base shear (32 % of the mast weight). These values correspond to the model 15.0-1-1.80 under the Kobe earthquake record. In the whole, for all the models and seismic records, the base shear yields values in a range of 4 to 30 % of the mast weight.
- As the IP and/or the MS vary, the dynamic properties of the system change. Furthermore, the dynamic answer appears amplified when some of the seismic records are used. This effect resulted more evident in the models with the minimum values of D and MS. Thus, the modification of the second moment area of the mast cross-section (proportional to the bending stiffness MS) as well as the IP can lead to an amplification of the response under certain type of the seismic acceleration records.
- From the FFT analysis of the displacement at the top of the mast, it is possible to observe that a peak in the range of 1 to 1.5 Hz is always present. Furthermore, as expected, the displacements are higher when the accelerograms exhibit a large frequency content in correspondence with this range. Thus, the fundamental frequency of the structure is one of the variables that have a significant influence on the dynamic pattern of the motion. Based on this fact, a rather good prediction of the top displacement was found using a simple SDOF model with its frequency matching the first eigenvalue of the structural system.
- Finally and as was mentioned before, the inherent features of each seismic record (PGA, frequency content, duration of the intense phase, etc.) have a considerable influence on the response. This finding deserves another study to assess the effect of the different characteristics of the earthquake records on the sensitivity of the structure. At present the authors address profuse study with a comprehensive database of earthquakes to assess this matter.

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Author Profile

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