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***Operational modal analysis of a self-supporting antenna mast***

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***Abstract:*** This work presents the results of an operational modal analysis on a self-supporting antenna mast and the comparison of these results with a finite element model. The FE model of the structure is built using SAP2000 software and the connection type between the different members composing the mast are modelled using three types of models: truss model, rigid model and mixed model. The structure is instrumented with accelerometers, at different setups for a detailed characterization of the mode shapes. In addition, two strain gauges are installed at the bottom of each leg of the tower. From the operational modal analysis, the natural frequencies and mode shapes of the self-supporting tower are obtained and these are compared with those calculated from the model. The results show that the best match with the experimental results is obtained for a mixed model, where the vertical columns are connected rigidly and the rest of the connections is assumed pinned. Also, low damping values are obtained, this is attributed to the low wind speed and the weather conditions during the measurements, which made the contribution of aerodynamic damping insignificant.

***Keywords:*** *Operational modal analysis; System identification; Model validation; Field test*

**1. Introduction**

Finite Element (FE) models predict the behavior of the structure, assuming that the model provides an accurate characterization of the response of the actual structure to the design loads considered. However, it is clear that this assumption is not always satisfied as models do not always accurately capture the response of the physical structure. Field tests are needed to assess the validity of such computational models, in particular for novel structural designs or unprecedented loading conditions.

System identification methods are frequently applied to identify experimental models that describe the actual behavior of a structure. In Operational Modal Analysis (OMA), the modal parameters (natural frequencies, mode shapes and modal damping ratios) are extracted from experimental models identified from output-only data. These can be readily compared to those computed based on the numerical model, in order to assess the quality of this model. The main advantage of OMA is that the structure is measured in operational conditions, avoiding the interruption of the normal operation of the structure during the test.

The first techniques for OMA were operating in the frequency domain and based on a power spectral density analysis. Over the past few decades, several effective and highly efficient output-only modal identification techniques have been developed, amongst which Frequency Domain Decomposition (FDD) (Brinker, Zhang et al. 2000) and Stochastic Subspace Identification (SSI) (Peeters and De Roeck 1999, Reynders 2012) are probably the most widely applied.

Many of the studies available in the literature have allowed confirming the validity of finite element models for the dynamic analysis of these structures by showing that the predicted modal parameters were close to those identified from the tests (Magalhaes, Caetano et al. 2008, Martins, Caetano et al. 2014, Celik, Tien Do et al. 2016). Once the validity of such a model has been shown, it can also be used to verify the actual loads acting on the structure, using indirect force identification techniques (Acampora, Macdonald et al. 2014, Feng, Sun et al. 2015, Maes, Iliopoulos et al. 2016, Maes, Reynders et al. 2016). This is particularly interesting when such loads cannot be directly measured, as is generally the case for dynamic wind loads.

Only a few ambient vibration tests on lattice towers have been reported in the literature, although such tests would be very welcome to assess various modelling assumptions. Peeters and De Roeck (Peeters and De Roeck 1999) have applied different system identification methods to obtain the modal characteristics of a 30 meters high self-supporting antenna mast. Kazemi Amiri and Bucher (Kazemi Amiri and Bucher 2017) investigated the feasibility of wind load reconstruction for a guyed mast. The modal parameters of the structure are first identified using OMA, after which they are used in the estimation of the modal wind loads. This study, however, does not explicitly involve a finite element model of the antenna mast.

According to the literature, one of the main assumptions that significantly affects the dynamic behavior predicted by an FE model of an antenna mast is the connection type between the different members composing the mast. Several papers discuss the difference between three types of models (see Figure. 1): truss models, where all connections are pinned, rigid models, where all connections are rigid, and mixed models, where the vertical columns are connected rigidly and the rest of the connections is assumed pinned (Da Silva, Vellasco et al. 2007, Taillon, Légeron et al. 2012, Santos Oliveira and Da Silva 2015, Lu, Ou et al. 2016). The majority of the researchers concludes that the mixed models are more accurate in describing the dynamic behaviour of the antenna masts, leading to a more accurate prediction in the design stage.

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| Figure. 1. Schematic overview of three different model types typically considered in FE modelling of antenna masts. (Own elaboration) |

The aim of this work is to present the results from an OMA on a self-supporting tower and to assess the validity of a detailed finite element model by a comparison of the natural frequencies and mode shapes, hereby considering the different model types presented in Figure. 1.

**2. Methodology**

The structure under consideration, shown in Figure 2, is a self-supporting steel antenna mast located in Matanzas, Cuba. The tower has a square cross-section and a total height of 58.8 m. From the bottom to a height of 38.5 m, the cross-section varies between 8.9 m by 8.9 m (bottom) to 1.2 m by 1.2 m (top). Between 38.5 m and 50.5 m, the tower has a constant cross section of 1.2 m by 1.2 m. Finally, a cantilever mast with a length of 8 m is placed at the top. The mast supports three types of antennas at different levels: FM, VHF, and UHF (Figure 2b).

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| (a) | (b) |
| Figure 2. (a) Schematic overview of the antenna mast and (b) overview picture. (Own elaboration) | |

The modal characteristics of the antenna mast are determined experimentally by means of operational modal analysis (OMA), starting from the acceleration and strain response of the structure under wind loading.

The response of the tower is measured using 10 accelerometers of type PCB 393B12 (sensitivity 10 V/g) and 8 conventional strain gauges of type Micro-Measurements CEA-06-250UN-350. The data acquisition is performed by means of a National Instrument (NI) data acquisition system, consisting of a NI CDAQ-9188 chassis equipped with four NI 9234 ICP modules for the acceleration measurements and two NI 9237 analog input modules for the strain measurements. A sampling frequency of 1651.6 Hz is adopted in the data acquisition. The measurements are performed in three setups, covering in total 24 measurement locations. The design of the measurement layout is based on the modal characteristics obtained from a preliminary finite element model of the structure. More details on the sensor layout can be found in (Luis García, Elena Parnás et al. 2020, Luis García, Maes et al. 2021).

## 2.1. System identification

The acceleration and strain signals are processed using the MACEC 3.3 Matlab toolbox for experimental and operational modal analysis, developed at the Structural Mechanics Section of KU Leuven (Reynders, Schevenels et al. 2014). Prior to the system identification, all signals are downsampled and highpass filtered. In the downsampling, the signals are first digitally lowpass filtered by means of a Chebyshev type I lowpass filter with a cut-off frequency of 6.61 Hz. The filter is applied in both the forward and the reverse direction to remove all phase distortion. The decimated signals are additionally highpass filtered by means of a digital fourth order Butterworth highpass filter with a cut-off frequency of 0.5 Hz, applied both in the forward and the reverse direction. The aim of this filter is to remove low-frequency components from the signals that are contaminated by measurement noise.

Figure 3 shows the power spectral density (PSD) of the accelerations obtained at the two highest measurement levels in setup 1 ( = 50.5 m and = 41.2 m) and Figure 4 shows the PSD of the corresponding strains measured at the bottom of the tower. Both the acceleration PSDs in Figure 3a and the strain PSDs in Figure 4 show clear peaks at frequencies 1.7 Hz, 3.1 Hz, and 5.6 Hz, corresponding to the natural frequencies of the antenna mast. When taking a closer look at the acceleration PSDs in Figure 3a, some additional peaks are observed at frequencies corresponding to multiples of 1.33 Hz and at 2.3 Hz. It is worthwhile noting that the acceleration PSDs in Figure 3b in contrast to those shown in Figure 3a do not show clear peaks at the resonance frequencies. Instead. they are dominated by the previously mentioned spurious harmonics multiples of 1.33 Hz and at 2.3 Hz. Converting the acceleration records into audio clips (.mp3) revealed that this disturbance was caused by interference with the radio transmission. Although this interference does not dominate the acceleration records for the top level (Figure 3a), this is unfortunately not the case for measurement levels  = 33.8 m and  = 41.2 m (Figure 3b), which are located very close to the radio transmission antennas. All channels corresponding to these two measurement levels have therefore been omitted in the system identification. In addition, three other acceleration channels have been omitted due to the presence of spurious low-frequency components. For each measurement level, three acceleration measurements have been retained. The strain signals are not contaminated by spurious harmonics and have all been included in the system identification.

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| (a)(b)  Figure 3. (a) Acceleration PSD for the tower top channels ( = 50.5 m) and (b) acceleration PSD for the channels at measurement level  = 41.2 m recorded during setup 1. The identified natural frequencies are indicated using vertical black dashed lines. Spurious peaks due to interference with the radio transmittance are indicated using vertical red dash-dotted lines. |

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| Figure 4. Strain PSD recorded during setup 1. The identified natural frequencies are indicated using vertical black dashed lines. |

For each of the three setups, the acceleration and strain signals are processed using the reference-based covariance-driven stochastic subspace identification algorithm (SSI-cov/ref, (Peeters and De Roeck 1999)) to extract the modal characteristics. Despite small interference with the radio transmittance, as previously was discussed, a very clear stabilization diagram is obtained for the three setups.

## 2.2 Setup combination

Due to the symmetry of the cross-section of the tower, the lowest bending modes occur in pairs, with modes in each pair characterized by nearly identical natural frequencies. In addition, it is found that the global bending directions for the modes obtained from the three setups are different, which is again due to the symmetry of the structure. Taking this into account, it is reasonable to assume that modes 1 and 2 obtained from setup 1 both consist of a linear combination of modes 1 and 2 obtained from setup 2 and in turn another linear combination of modes 1 and 2 obtained from setup 3. The same holds for modes 3 and 4. The third order bending modes appear to be uncoupled. This is most probably due to the fact that the higher modes are more sensitive to local details which perturb the global symmetry. In this case, only one of the two modes (i.e. mode 5) could be identified. The fact that different bending directions are obtained from the three setups complicates the assembly of the global mode shapes. The coupling of the first two pairs of bending modes is explicitly accounted for. The mode shapes for setups 2 and 3 are mapped onto the modes obtained for setup 1 in a least-squares sense. The natural frequencies and modal damping ratios of both pairs of bending modes are averaged. For the same reason, the global bending directions of the modes after combination of the setups are also different from those obtained from the finite element model.

Table 1 shows the results after combination of the setups. Figure 5 shows the corresponding mode shapes. For plotting purposes, the displacement mode shapes have been extrapolated towards the corners of the square cross section of the tower at each measurement level, using the mode shape at three points and assuming a rigid diaphragm. Modes 1 to 4 are two pairs (1, 2) and (3, 4) of respectively first and second order bending modes of the tower. Mode 5 is a third order bending mode.

Table 1. Identified natural frequencies , modal damping ratiosMPC (modal phase collinearity), MP (mean phase), MPD (mean phase deviation), and description of the mode shapes.

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| Mode |  |  | MPC | MP | MPD |
| 1 | 1.75 | 0.27 | 1.00 | 1.14 | 1.09 |
| 2 | 1.75 | 0.27 | 0.89 | -10.65 | 9.70 |
| 3 | 3.10 | 0.26 | 1.00 | -0.10 | 1.65 |
| 4 | 3.10 | 0.26 | 0.99 | 2.00 | 1.91 |
| 5 | 5.43 | 0.21 | 0.99 | 0.90 | 1.96 |

Table 1 presents the identified natural frequencies, modal damping ratios, the modal phase collinearity of the mode shape (MPC), the mean phase of the mode shapes (MP), and the mean phase deviation of the mode shape (MPD,), after normalization of the mode shape to unit maximum displacement. The MPC, MP, and MPD values listed in Table 1 can be used to assess the quality of the identified modes. Indeed, if real normal modes are expected, as is the case for proportionally damped structures, the MPC, MP, and MPD values should approximate 1, 0° and 0°, respectively (Reynders, Schevenels et al. 2014). In general, very high MPC values and relatively low MP and MPD values are obtained.

Figure 6 presents the auto-MAC matrix of the identified mode shapes. The large off-diagonal values observed for mode pairs (1, 3) and (2 4) indicate a large similarity of the mode shapes, also observed from Figure 5. In addition, mode 5 shows some similarity with mode 2 (MAC = 0.68). Although the measurement setup was originally designed to enable a clear distinction between the different modes of interest, the fact that several channels have been disregarded because of poor signal quality has significantly complicated the distinction capacity. Finally, note the low auto-MAC values between modes 1 and 2 and between modes 3 and 4. These low values confirm the orthogonality between the identified bending modes.

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| (a) Mode 1 – 1.75 Hz | (b) Mode 2 – 1.75 Hz |
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| (c) Mode 3 – 3.10 Hz | (d) Mode 4 – 3.10 Hz |
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| (e) Mode 5 – 5.43 Hz | |
| Figure 5. Identified displacement mode shapes (after extrapolation) and strain mode shapes for modes 1 - 5. | |

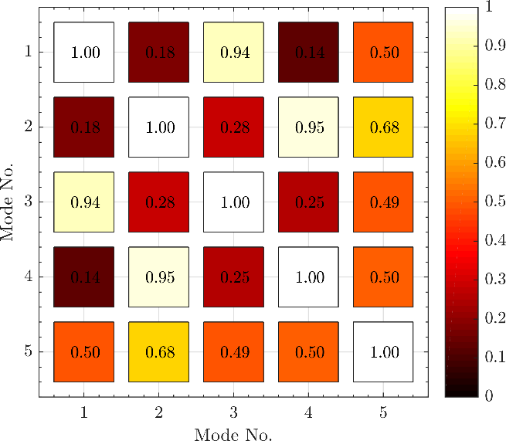


Figure 6. Auto-MAC values for the identified mode shapes after combination of the setups.

# **3. FE model validation**

A detailed finite element model of the antenna mast is constructed using the finite element software package SAP2000. The geometry of the mast is obtained from blueprints of the structure. For all steel members, a Young’s modulus of 200 GPa, a Poisson ratio of 0.3, and a density of 7800 kg/m3 are used. The four legs are assumed clamped at the foundation. For the connections between the different beam members composing the columns and the braces, the three different topologies discussed and summarized in Figure. 1 are separately considered: (1) a truss model with pinned connections, (2) a completely rigid model, and (3) a mixed model with rigid columns and pinned braced members. For each of the three models, the calculated modal characteristics (natural frequencies and mode shapes) are compared to those obtained from the measurements. Since the lowest bending modes of the FE models (modes 1, 2, 3, and 4) also occur in pairs, the mapping procedure described before was also adopted here to match the bending directions to those present in the OMA results. The OMA modes are hereby adopted as the reference in the mapping.

Table 2 compares the identified modal characteristics to those obtained from the three different FE models. For the truss model, only modes 3 and 4 can be properly matched, hereby adopting a threshold . For modes 1, 2, and 5, it was not possible to find the corresponding modes, indicating a large discrepancy between the model prediction and the actual observed structural behavior. For the rigid model, all five modes are matched and very high MAC values are obtained. The rigid model, however, overestimates the natural frequencies, indicating a too stiff behavior of the model. For the mixed model, finally, a very good overall agreement with the measured modal characteristics is observed, both in terms of the natural frequencies and the mode shapes, indicating a very good overall prediction of the dynamic structural behavior. Although the natural frequencies are still slightly overestimated, the model is deemed sufficiently accurate from a design point of view.

Table 2. Comparison of the identified modal characteristics with those obtained from the three different FE models (truss, rigid, and mixed).

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| Mode  No. | Natural frequency | | | | Difference | | | MAC | | |
| OMA | FEM | | | OMA/ Truss | OMA/ Rigid | OMA/Mixed | OMA/ Truss | OMA/ Rigid | OMA/  Mixed |
| Truss | Rigid | Mixed |
| 1-2 | 1.75 | - | 1.87 | 1.81 | - | 6.42 | 3.32 | - | 1.00 | 1.00 |
| - | 0.95 | 0.97 |
| 3-4 | 3.10 | 3.81 | 3.34 | 3.25 | 18.64 | 7.19 | 4.62 | 0.99 | 0.99 | 0.99 |
| 0.98 | 0.98 | 0.97 |
| 5 | 5.43 | - | 5.84 | 5.70 | - | 7.02 | 4.74 | - | 0.98 | 0.95 |

Figure 7 shows mode shapes obtained from the mixed FE model. As previously explained, modes 1 – 4 consist of two pairs (1, 2) and (3, 4) of respectively first and second order bending modes of the tower. Mode 5 is the third order bending mode. In contrast to modes 1 – 4, this bending mode does not occur in pair. Figure 9 compares the measured and the predicted strain mode shapes at the bottom of the tower (predictions based on the mixed FE model). Especially for modes 1 and 2, a very good agreement is observed. The fact that these strains are accurately predicted constitutes very important design feedback, since the strains in the legs of the tower play an important role and the lower modes are the ones that are most dominantly excited by wind loading. For modes 3 and 4, a larger discrepancy is observed. This may be due to identification inaccuracies as well as the influence of local details which for higher modes start to play a more important role. For mode 5, however, a very good agreement between the predictions and the mixed model is again observed.

These results show that the natural frequencies and mode shapes of the tower can be accurately predicted by a FE model, provided the actual connections between the members are considered. Equally important for the response to wind loads is the damping of the tower which cannot be easily modelled based on physical grounds. In the remainder of this section, the damping values obtained from the OMA (Table 2) are interpreted and related to what is described in the literature and prescribed by the design codes.

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| 1. Mode 1 – 1.81 Hz | | | 1. Mode 2 – 1.81 Hz | | 1. Mode 3 – 3.25 Hz | | |
| C:\4katia_proces\OMA PAPER\pictures\sap_mode4.JPG | |  | |  | |  | |
| 1. Mode 4 – 3.25 Hz | | | | 1. Mode 5 – 5.70 Hz | | | |
| Figure 7. Mode shapes obtained from the mixed FE model. The horizontal section is taken at 16 m from the bottom of the tower. | | | | | | | |

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| (a) Mode 1 | (b) Mode 2 | (c) Mode 3 |
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| (d) Mode 4 | (e) Mode 5 |  |
| Figure 8. Comparison of the identified (blue) and predicted (red) strain mode shapes. The predictions correspond to the mixed FE model. | | | |

The estimated damping accounts for multiple independent damping mechanisms, including material damping, friction in the joints, aerodynamic damping, and radiation damping at the base. Although the focus of this work is not on the distinction between the different sources of damping, it can be reasonably assumed that the contribution of aerodynamic damping in this case is low, taking into account the very low wind speeds during the measurements (estimated less than 10 m/s according to the meteorological report) (Glanville, Kwok et al. 1996, Peeters and De Roeck 1999). This may also explain the very low modal damping values obtained for each of the five bending modes (about 0.20 to 0.30%). The influence of the wind velocity on the damping is reported in (Ballio, Maberini et al. 1992, Glanville and Kwok 1995, Calotescu and Solari 2016). Ballio et al. (Ballio, Maberini et al. 1992) obtained (total) damping values of 0.68 - 0.9 % under a wind velocity of 12.5 m/s for a 100 m tall self-supporting tower in Milan. Glanville and Kwok (Glanville and Kwok 1995) studied a 67 m tall self-supporting tower in Australia and obtained values of 0.25, 0.75 and 1% for the three first bending modes under a wind velocity of 20 m/s. Finally, Calotescu and Solari (Calotescu and Solari 2016) have performed an analytical study for a 50 meters self-supporting tower, where the aerodynamic damping is compared to the structural damping as a function of the wind speed. For a wind speed of 30 m/s, a total damping ratio of 1.2 % is obtained, where the aerodynamic damping was found to be much higher than the assumed structural damping (0.5%).

Finally, under the assumption that aerodynamic damping is negligible in the considered case, it is noticed that the identified damping ratios are generally much lower than the structural damping prescribed by the design codes. For example, design code ASCE 7-10:2010 (Institute 2010) proposes a damping value of 0.5 – 1.2%. Eurocode 1 (EC8:Part1 2005) provides 0.5% and 0.3% for bolted and welded connections, respectively, while DIN 4131 (1055-4:1977-05 2002) propose 0.5 – 0.8% and 0.3%, respectively. More research is planned in order to verify this behaviour for other towers and to investigate the influence of aerodynamic damping for higher wind speeds.

# **4. Conclusions**

This paper presents the results of an extensive operational modal analysis on a self-supported 50 m tall antenna mast in Cuba, the main conclusions are:

1. The acceleration measurements in the vicinity of the radio antennas are all significantly polluted by electrical noise. In order to facilitate the modal analysis, sensors close to the antennas have been omitted. Furthermore, it was found that the lowest bending modes of the tower occur in pairs, which complicates the mode matching. In order to overcome this issue, a projection of the mode shapes has been performed prior to the combination of the modes.
2. The identified natural frequencies and modes shapes are compared to those predicted by three different types of finite element models that are commonly adopted in the modelling of antenna masts, and which distinguish in the connections between the steel members composing the tower. It is found that the best match with the experimental results is obtained for a so-called mixed model, where the vertical columns are connected rigidly and the rest of the connections is assumed pinned.
3. The identified modal damping ratios range between 0.21% and 0.27 %, which is very low. The low damping values, however, are attributed to the low wind speed and the weather conditions during the measurements, which made the contribution of aerodynamic damping insignificant.

5. References

1055-4:1977-05, D. (2002). "Torres de acero portadoras de antenas." Alemania.

Acampora, A., J. H. Macdonald, C. Georgakis and N. Nikitas (2014). "Identification of aeroelastic forces and static drag coefficients of a twin cable bridge stay from full-scale ambient vibration measurements." Journal of Wind Engineering and Industrial Aerodynamics 124: 90-98.

Ballio, G., F. Maberini and G. Solari (1992). "A 60 year old, 100 m high steel tower: .limit states under wind actions." Journal of Wind Engineering and IndustrialAerodynamics 41: 2089-2100.

Brinker, R., L. Zhang and P. Andersen (2000). Modal Identification from Ambient responses using frequency Domain Decomposition. Proceedings of the International Modal Analysis Conference (IMAC). USA: 625-630.

Calotescu, I. and G. Solari (2016). "Alongwind load effects on free-standing lattice towers." Journal of Wind Engineering and Industrial Aerodynamics 155: 182-196.

Celik, O., N. Tien Do, O. Abdeljaber, M. Gul, O. Avci and F. N. Catbas (2016). Recent Issues on Stadium Monitoring and Serviceability: A Review. Conference Proceedings of the Society for Experimental Mechanics Series.

Da Silva, J., P. Vellasco, S. de Andrade and L. de Lima (2007). "Structural Analysis of Guyed Steel Telecommunication Towers for Radio Antennas." Journal of the Brazilian Society of Mechanical Sciences and Engineering 29(2).

EC8:Part1 (2005). Eurocode 8: Design of structures for earthquare resistance - Part 1: General rules, seismic actions and rules for builidings.

Feng, D., H. Sun and M. Feng (2015). "Simultaneous identification of bridge structural parameters and vehicle loads." Computers and Structures 157: 76-88.

Glanville, M. and K. Kwok (1995). "Dynamic characteristics and wind induced response of a steel frame tower." journal of Wind Engineering and Industrial Aerodynamics 54: 133-149.

Glanville, M., K. Kwok and R. Denoon (1996). "Full-scale damping measurements of structures in Australia." Journal of Wind Engineering and Industrial Aerodynamics 59: 349-364.

Institute, S. E. (2010). ASCE 7-10: American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures. USA.

Kazemi Amiri, A. and C. Bucher (2017). "A procedure for in situ wind load reconstruction from structural response only based on field testing data." Journal of Wind Engineering and Industrial Aerodynamics 167 75-86.

Lu, C., Y. Ou, M. Xing and J. Mills (2016). "Structural Analysis of Lattice Steel Transmission Towers: A Review." Journal of Steel Structures & Construction 2(1).

Luis García, K., V. Elena Parnás, K. Maes and G. Lombaert (2020). "Mediciones a escala real de torre autosoportada empleando acelerómetros y strain gauges." Ingeniería y Desarrollo. Universidad del Norte. 38(1): 259-278.

Luis García, K., K. Maes, V. Elena Parnás and G. Lombaert (2021). "Operational modal analysis of a self-supporting antenna mast." Journal of Wind Engineering and Industrial Aerodynamics 209.

Maes, K., A. Iliopoulos, W. Weijtjens, C. Devriendt and G. Lombaert (2016). "Dynamic strain estimation for fatigue assessment of an offshore monopile wind turbine using filtering and modal expansion algorithms." Mechanical Systems and Signal Processing 76: 592-611.

Maes, K., E. Reynders, A. Rezayat, G. De Roeck and G. Lombaert (2016). "Offline synchronization of data acquisition systems using system identification." Journal of Sound and Vibration 381: 264-272.

Magalhaes, F., C. Caetano and A. Cunha (2008). "Operational modal analysis and finite element model correlation of the Braga Stadium suspended roof." Engineering Structures 30: 1688–1698.

Martins, N., E. Caetano, S. Diord, F. Magalhães and Á. Cunha (2014). "Dynamic monitoring of a stadium suspension roof: Wind and temperature influence on modal parameters and structural response." Engineering Structures 59: 80-94.

Peeters, B. and G. De Roeck (1999). "Reference-based stochastic subspace identification for output-only modal analysis." Mechanical Systems and Signal Processing 13(6): 855-878.

Reynders, E. (2012). "System Identification Methods for (Operational) Modal Analysis: Review and Comparison." Archives of Computational Methods in Engineering 19(1): 51-124.

Reynders, E., M. Schevenels and G. De Roeck (2014). MACEC 3.3: A MatLab toolbox for experimental and operational modal analysis. Faculty of engineering, Department of civil engineering, Structural Mechanics Section, Leuven, Belgium.

Santos Oliveira, B. and J. Da Silva (2015). Análise dinâmica de torres de aço de telecomunicações submetidas à ação não determinística do vento. XXXVI Iberian Latin-American Congress on Computational Methods in Engineering, Rio de Janeiro, RJ, Brazil.

Taillon, J., F. Légeron and S. Prud'homme (2012). "Variation of damping and stiffness of lattice towers with load level." Journal of Constructional Steel Research 71: 111-118.