Asociación Argentina de Mecánica Computacional

Mecánica Computacional Vol XXXII, págs. 291-305 (artículo completo)
Carlos G. García Garino, Aníbal E. Mirasso, Mario A. Storti, Miguel E. Tornello (Eds.)
Mendoza, Argentina, 19-22 Noviembre 2013

DAMAGE EVALUATION AND MODEL UPDATING OF A RETROFITTED REINFORCED CONCRETE FRAME WITH G-FRP.

Francisco A. Calderón[a], Gustavo L. Palazzo[a], Arnaud Deraemaeker[b] and Roberto Aguiar Falconi[c]

[a]*CeReDeTeC, National Technological University, Rodriguez 273, 5500, Mendoza, Argentina. francisco.calderon@frm.utn.edu.ar, gpalazzo@frm.utn.edu.ar*

[b]*BATir, Free University of Brussels, Avenue F. Roosevelt 50 Brussels, Belgium. aderaema@ulb.ac.be, http://batir.ulb.ac.be*

[c]*Scientific Research Center, Army Polytechnic School, Av. Gral. Rumiñahui s/n, Sangolquí Ecuador. rraguiar@espe.edu.ec, http://www.espe.edu.ec*

Keywords: System identification, Model updating, Structure retrofitted, Dynamic loads.

Abstract. This paper considers basic system identifications techniques, without measurement of the excitation, in a reinforced concrete frame. The objective is to obtain experimental parameters, which permit to update a numerical model. The paper was development in three stages. Tests of the frame in a shaking table were the first stage. The frame was subject to dynamic actions, and then was retrofitted with fiber-reinforced polymers. Afterwards, it was again subject to dynamic actions. The system identifications tests were the second stage. With simple methodologies, frequencies and damping rates were obtained in the structure with and without retrofitting, after different amplitudes of the dynamic actions. And the last stage was about the model updating. A parameter of the finite element model of the structure was updating until the numerical response was close the experimental response. In the conclusions, the possibilities of the system identification technique applied to obtain reliable numerical model are discussed.

1 INTRODUCTION

The finite element method is employed for static and dynamic analysis of structural systems in a massive form. In structural dynamic, often is required to obtain the modal properties of the system analyzed (natural frequencies, modal forms and modal damping), for which the finite element method is regularly used.

However, development of reliable techniques of vibrations measure allowed that these properties are obtained experimentally from which there has been considerable disagreement among the predictions of the finite element models and their experimental counterparts.

Therefore research has focused on finding ways to leverage the benefits of the finite element method and experimental techniques simultaneously.
The result is the *Finite Element Model Updating*, through which it seeks to generate reliable models through parametric adjustments on the same, so as to minimize the difference between the analytical predictions and experimental results (Velez et al. 2009).

In the particular case of the analysis of the dynamic behavior of elastic-mechanical systems, the modal properties calculated using the finite element method often differs from those obtained by vibration tests. Although experimental data has errors that can cause a low correlation between these and predictions, the model fitting theory assumes that the main causes of disagreement are the theoretical and experimental errors in the model. Friswell and Mottershead (1993) consider three types of errors:

i. Errors in the model structure, which occur when there is uncertainty in the governing physical equations (as in the modeling of systems with strong nonlinear behavior).

ii. Errors in the parameters of the model, within which is the application of inappropriate boundary conditions, as well as inaccurate assumptions used to simplify the model.

iii. Errors in the order of the model, whose occurrence is due to the discretization of complex systems, which can produce enough order models.

The purpose is to modify model fitting parameters of mass, stiffness and damping of the numerical model to obtain a better agreement between the numerical results and experimental information. If the model in question has been developed for purposes of prediction, the correlation should be improved correcting dubious assumptions and not doing unphysical meaning modifications (Friswell and Mottershead 1993, Ahmadian et al. 1998).

The article Friswell and Mottershead (1993) is an essential reference on the techniques available to update finite element models. The document provides descriptions of the methods that were available at the time of its preparation, in particular, direct methods and iterative methods based on sensitivity data with the use of modal and frequency response functions.

After the appearance of the article (Friswell and Mottershead 1993), there have been few similar documents and research has focused on the development of new methodologies and performance evaluation thereof. However, some items of its content are dedicated to literature reviews. Dunn (1994) studied the methods of adjustment and its relationship to the number of unknowns, aspects of uniqueness in the solutions and the advantages and disadvantages of using modal and FRF data. Hemez and Doebling (2001) reviewed the state of the art of updating models applied to nonlinear systems. Datta (2002) presented a review of direct methods of renovation and introduced two alternative approaches based on partial allocation eigenestructures and embedded eigenvalues. Calvi (2005) pointed out the problems associated with traditional deterministic approach and stochastic approach described the adjustment and validation of models. Ahmadian et al. (1998) reviewed different regularization techniques and its application to parameter estimation model updating.

In this paper a model updating of a prototype tested in a shaking tables is considered. The updating is based on dynamic parameters obtained by some simple system identification.

The prototype is a 3D one level reinforced concrete frame, subject to several dynamic actions. The frame was severely damaged and then, it was retrofitted with glass fiber-reinforced polymers (G-FRP). Several dynamic actions were newly applied. The frame and the tests are described in the 2nd section.

There are various system identifications techniques to obtain dynamic parameters in structures. But in this paper only two basic methodology are considered in the 3rd section, without the measurement of the input. With them, frequencies and damping rates are obtained for different level of the actions applied.

A finite element model of the frame, with and without G-FRP, was made. It is presented in the 4th section. The initial mechanics parameters were taken of the tests on the frame's materials. Then, an updating was done, based on the frequencies measurement in the dynamic tests. With the last updating, the time history of displacement was obtained; and it was compared with the time history measured experimentally. In the conclusions (the 5th section), the possibility to update models with basic system identifications techniques are highlighted.

2 PROTOTYPE STUDIED

2.1 Geometrical and mechanicals characteristics of the prototype

The 3D one level reinforced concrete frame, the prototype, is shown in the Figure 1. It was retrofitted with G-FRP. It had 2.15 m in the longitudinal directions, 1.625 m in the transversal direction, and 2.75 m of high over its supports.

It had a slab with 0.08 m of thickness, with two masses of 9.0 kN each one. The dimensions of the columns were 0.15 x 0.125 m, and the beams had 0.20 x 0.125 m. The concrete had a specified compressive strength of 20 MPa, and the reinforcement had 420 MPa of specified yield tension. The G-FRP was SikaWrap Hex-100G. The general geometrical parameters are given in Figure 2, and the details of the cross section are shown in Figure 3.

a) Without G-FRP

b) With G-FRP

Figure 1: Prototype tested.

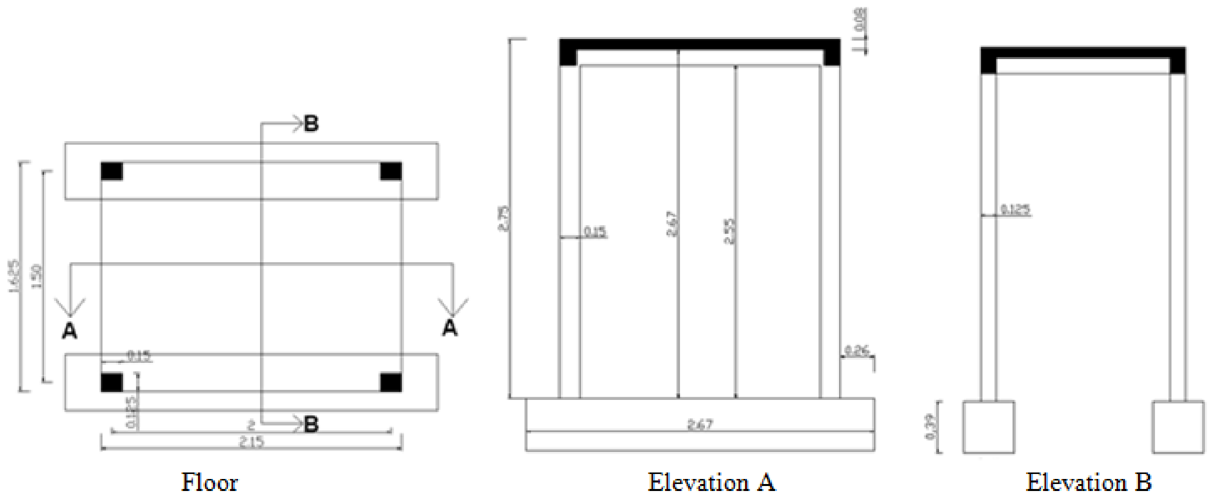


Figure 2: Global dimensions of the prototype.

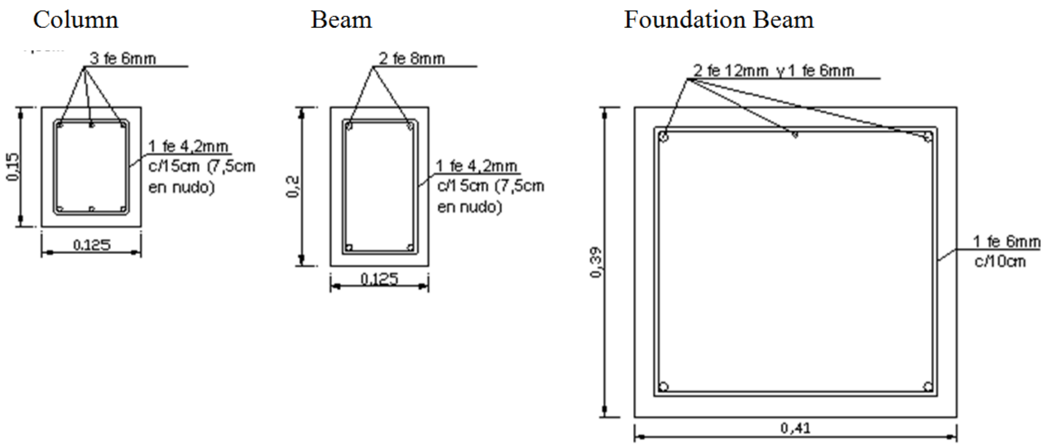


Figure 3: Cross sectional dimensions of the prototype's members.

2.2 Characteristics of the tests in the shaking table

The frame was subjected to dynamic tests (mix of sine waves) in the shaking table of the National University of San Juan (Argentina). Actions were only applied in the longitudinal directions. The original frame was subjected to 5 mix waves, with increasing acceleration amplitude from 0.91 m/s2 (first test) to 3.12 m/s2 (fifth test). The maximum displacement measurement was 64.88 mm (sixth test). The retrofitted frame was subjected to 5 mix waves, with increasing acceleration amplitude from 0.98 m/s2 (first test) to 4.99 m/s2 (fifh test). In this case, the maximum displacement measurement was 39.88 mm (fifth test).

Two LVDT was employed to measure displacement in the metallic diagonals of the frame (they can be seen in Figure 1-a), two accelerometers were placed in the slab and one in the basis of the shaking table.

More details about the frame and the test fulfilled are in Palazzo et al., 2012.

2.3 Damage evaluation

The damage evaluation was analyzed in the frame (original and retrofitted), based on the degradation of the lateral stiffness. The parameter was calculated as the approximate gradient of the lateral force vs. displacement in the slab curve. The stiffness was measured with test data. The displacement was measured in the frame diagonal with LVDT as seen in Figure 1.a, the force was computed as total mass multiplied by the acceleration (step by step). Then we estimated the stiffness through the secant.

In Figure 4 it is possible to see the reduction of the stiffness in each test. In the original frame there was an important reduction of the stiffness after the fifth test (50%). In the retrofitted frame this reduction was smaller (7%). Figure 5 is showing the reduction of the stiffness versus the peak of acceleration in the shaking table.

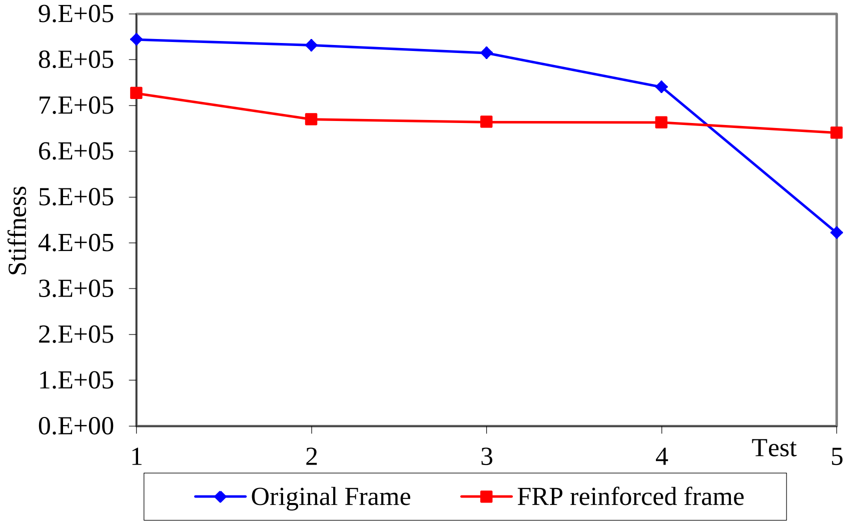


Figure 4: Evolution of the damage in the prototype.

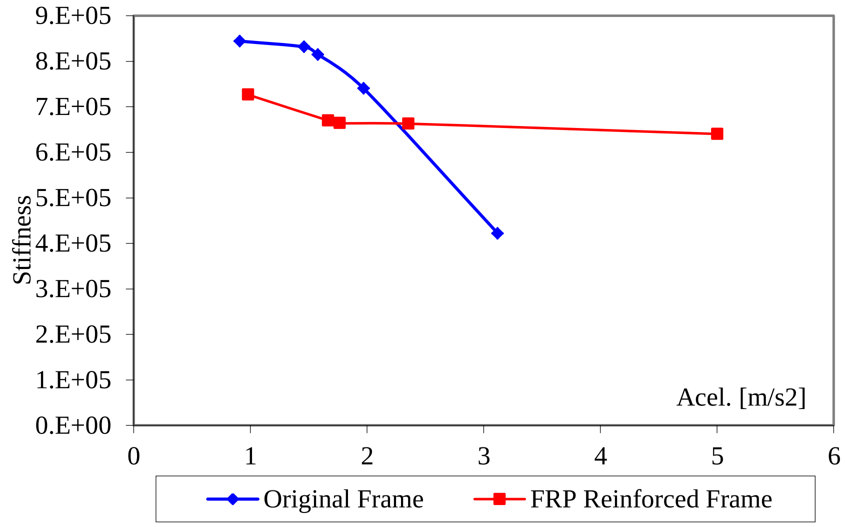


Figure 5: Evolution of the damage in the prototype.

3 SYSTEM IDENTIFICATION

3.1 Characteristic of the dynamic tests

Two basic system identifications techniques were considered to obtain dynamic parameters of the structure: the resonant amplification method (Clough and Penzien, 1975), and the pick method (Bishop and Gladwell, 1961).

Two Terra accelerometers (Weir-Jones Grup) were considered for the first method; and three accelerometers Basalt (Kinimetrics) were employed for the second method (one of them is shown in Figure 6). In both cases, the accelerometers were placed on the frame's slab.

Figure 6: Accelerometers used in the dynamic tests.

3.2 Resonant amplification method

After the first and the last dynamic tests on the frame with and without G-FRP, a frequency-response curve was done. A harmonic excitation was applied to the frame, with different frequencies, and the displacement amplitudes were measured in the slab. Then, it is possible to draw the frequency-response curve. It is shown in Figure 7 for the retrofitted structure, after the first dynamic test. The dynamic magnification factor Ra for any given frequency, is the ratio of the response amplitude at that frequency to the zero-frequency (static) response. The pick response permits to obtain the frequency of the structure. Also, with the frequency-response curve is possible to apply the half-power method to get the damping ratio ξ (Clough and Penzien, 1975).

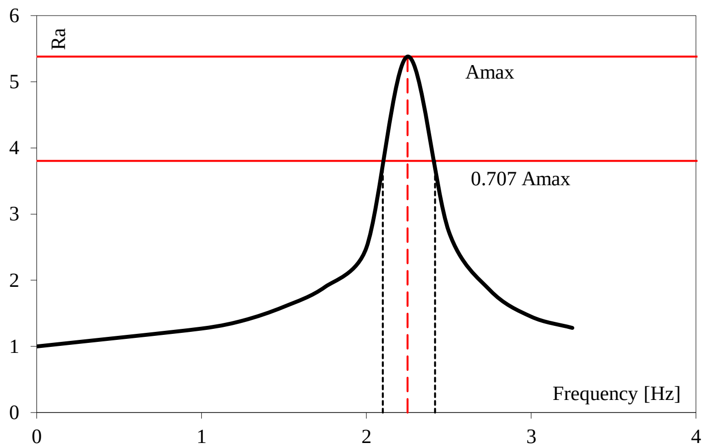


Figure 7: Frequency-response curve.

The frequencies and damping obtained with the frequency-response curves are given in Table 1. It is observed that the frequency in the original frame had a reduction of 45.5% between the first and the fifth test. This reduction is 6.7% for the retrofitted frame. These reduction percentages are similar to values given in the damage evaluation section. The damping rate had an increase of 120% in the original frame, and had not variation in the G-FRP frame.

Pórtico	Ensayo	Frec. [Hz]	ξ [%]
Original	1º	2.75	4.00
	5º	1.50	8.80
Reforzado	1º	2.25	7.00
	5º	2.10	7.00

Table 1. Frequencies and damping rates according to resonant amplification method.

3.3 Peak picking method

The pick method was applied in the G-FRP frame, after the last dynamic test. In this method it is necessary to get the Fourier spectrum of the acceleration time history measurement in the accelerometers. In this paper this spectrum was obtained with the Fast Fourier Transform (FFT). The pick in the amplitudes of the Fourier spectrum vs. frequencies curve permits to obtain the frequencies of the structure.

The acceleration time history measurements were obtained with the Basalt accelerometers in the frame's slab with two parameters: i- 25 records, with 200 Hz during 1 minute, and ii- 9 records, 250 Hz during 20 minute. The average Fourier spectrums are shown in Figure 8 and 9 and are computed with the software SMA of Kinimetrics.

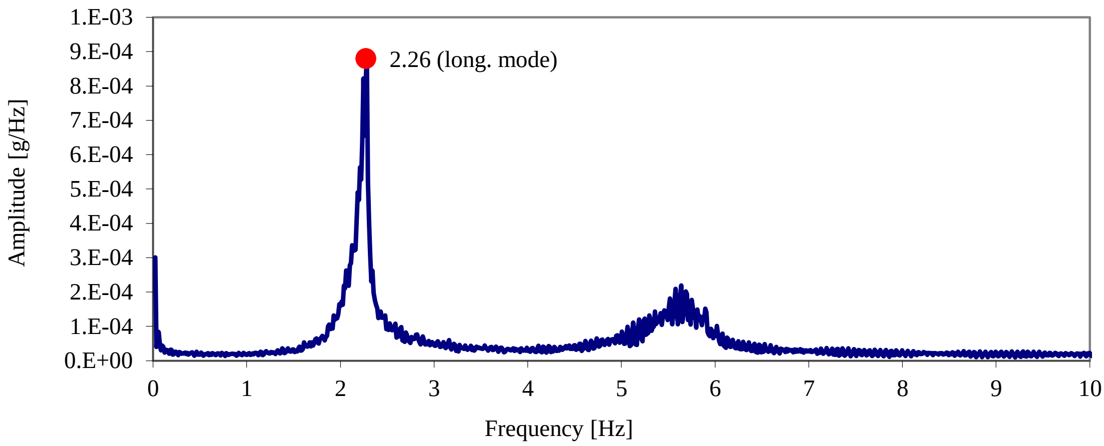


Figure 8: Average Fourier spectrum in the G-FRP frame (25, records with 200 Hz during 1 minute).

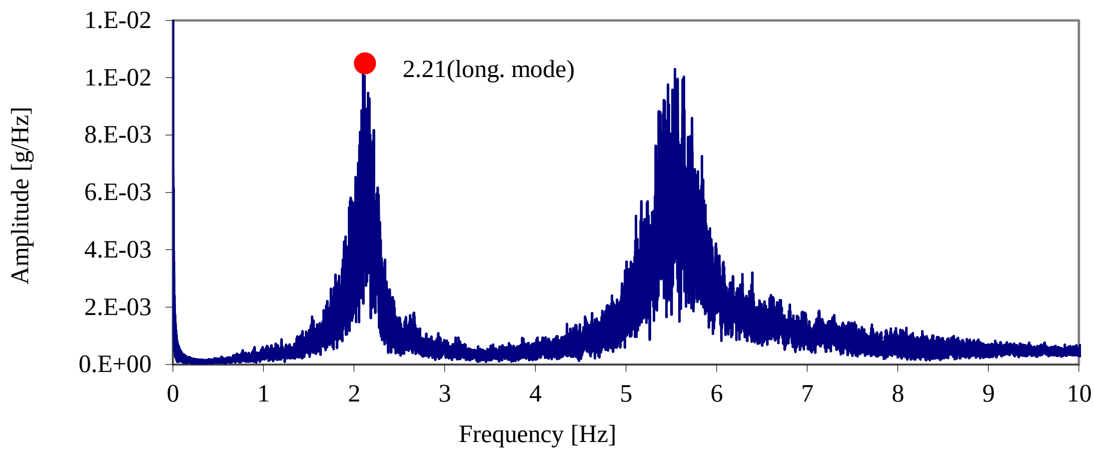


Figure 9: Average Fourier spectrum in the G-FRP frame (9 records, 250 Hz during 20 minute).

The frequencies with the pick method are given in Table 2, the average values μ and the standard deviation values σ are given. The difference between the frequencies measurement with the two kinds of records is of 2.21%. If the G-FRP frame's frequencies after the last test are compared (Table 1 y 2), the difference between the resonant amplification method and the pick method is of 5.2%.

Test	Sampling Rate [Hz]	Records	Record Length [min]	μ [Hz]	σ^2 [Hz]
1	200	25	1	2.26	0.03
2	250	9	20	2.21	0.07

Table 2. Frequencies according to pick method.

4 MODEL UPDATING

4.1 FEM's model

For the numerical simulations was used the commercial software Seismostruct, the same is free for academic purposes.

This element finite software was developed for numerical simulations of different type of structures subjected to static and dynamic loads (such as ground motion). Seismostruct is capable to predict the structure performance under large displacement, taking into account both geometric nonlinearities and material inelasticity.

In the Software, use is made of the so-called fiber approach to represent the cross-section behavior, where each fiber is associated with a uniaxial stress-strain relationship; the sectional stress-strain state of beam-column elements is then obtained through the integration of the nonlinear uniaxial stress-strain response of the individual fibers in which the section has been subdivided. In the considered model in this paper (Figure 10), the cross sections were subdivided in 100 fibers and in the plastic hinge zone in 200 fibers (Figure 11).

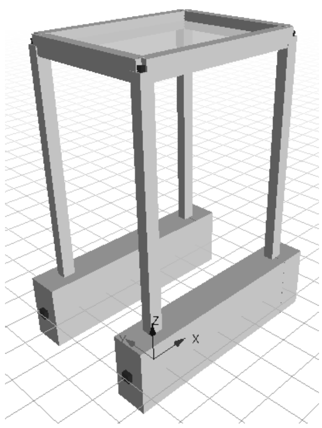


Figure 10: FEM of the frame in SeismoStruct.

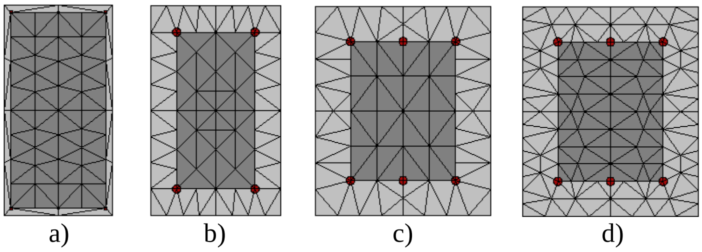


Figure 11: Discretization of Different Elements; a) Foundation Beam (100 fibers), b) Beams (100 fibers), c) Columns (100 fibers) y d) Plastic Hinge zone on Columns (200 fibers).

The steel model is a uniaxial bilinear stress-strain model with kinematic strain hardening, whereby the elastic range remains constant throughout the various loading stages, and the kinematic hardening rule for the yield surface is assumed as a linear function of the increment of plastic strain. The concrete model is an uniaxial nonlinear constant confinement model, initially programmed by Madas (1993) that follows the constitutive relationship proposed by Mander et al. (1988) and the cyclic rules proposed by Martinez-Rueda and Elnashai (1997). The confinement effects provided by the lateral transverse reinforcement are incorporated through the rules proposed by Mander et al. (1988) whereby constant confining pressure is assumed throughout the entire stress-strain range.

This is the constant confinement factor, defined as the ratio between the confined and unconfined compressive stress of the concrete, and used to scale up the stress-strain relationship throughout the entire strain range. Although it may be computed through the use of any confinement model available in the literature, the use of the Mander et al. (1988) is recommended by Seismostruct. Its value usually fluctuates between the values of 1.0 and 2.0 for reinforced concrete members and between 1.5 and 4.0 for steel-concrete composite members.

Material	Parameter	Magnitud
Reinforced	Modulus of elasticity E [MPa]	200000
	Yield strength f_y [MPa]	420
	Strain hardening parameter μ	0.005
	Fracture/buckling strain ε_{ult} [mm/mm]	0.1
	Specific weight γ_a [kN/m^3]	78
Concrete	Compressive strength f_c [MPa]	Variable
	Tensile strength f_t [MPa]	0.0
	Strain at peak stress ε_c [mm/mm]	0.002
	Confinement factor k_c	Variable
	Specific weight γ_c [kN/m^3]	24

In Seismostruct is possible to implement the frame element with two formulations: one classic based on displacement (called infrmDB) and another latest based on forces (infrmFB). In the software, it is indicated that simulated inelastic materials with the classical formulation is necessary to refine the discretization. Thus the forced-based formulation is the better option. Based on the foregoing, for the simulations of the frame without and with reinforcement was used, force based formulation (infrmFB).

Rectangular solid section was adopted for beams and columns. Dynamic time-history loads were considered, and then, dynamic time-history analysis was used to predict the nonlinear inelastic response of the frame subjected to dynamic actions.

The Figure 12 shows the input acceleration records for the first set of dynamic tests, and the Figure 13 shows the input records for the second set of tests where the frame was repaired with Glass-FRP in plastic hinge zone.

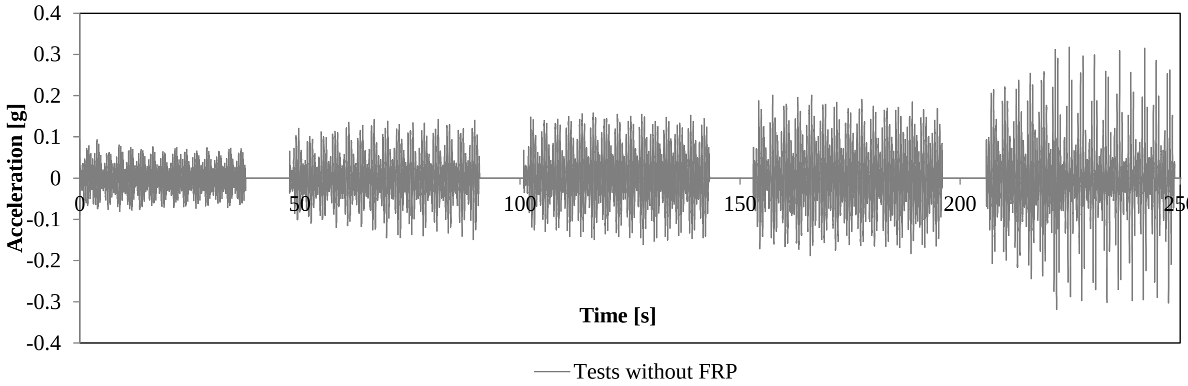


Figure 12: Input acceleration records for the original frame.

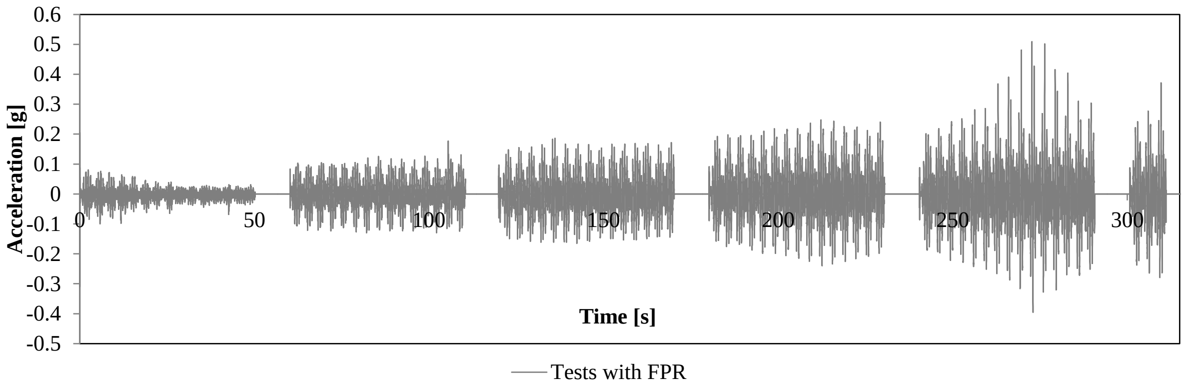


Figure 13: Input acceleration records for the retrofitted frame with G-FRP.

4.2 Model updating methodology

The model updating was based on: i- the frequencies measured during the dynamic tests and, ii- the modification of the Young's modulus E for the concrete model. For the model updating of the first model of the frame with and without G-FRP was applied the equation 1.

$$E_{new} = \frac{E_{\mathrm{mod}}}{\rho} \qquad (2)$$

where ρ: $(T_{mesured}/T_{initial\ model})^2$, with T as a period.

The updating was done in the frame with and without G-FRP, only for the first and the last test (where the frequencies were measured). The methodology applied was explained in the follow paragraphs:

. Frame without G-FRP after the first test: i- First model: The E obtained in test was considered (E = 21.02 GPa), then the numerical and the experimental frequencies were compared; ii- Updated 1: The E was reduced until the numerical and the experimental frequencies were approximately the same (E = 15.33 GPa).

. Frame without G-FRP after the fifth test: i- Updated 1: Model idem to the last case; ii-

Updated 2: The E = 15.33 GPa was considered in the frame, except in the last 0.40 m of the columns (zone with several cracks), there the E was 0.66 MPa.

. Frame with G-FRP after the first test: i- Updated 2: Model idem to the last case, where the G-FRP was considered enlarging the confinement in the last 0.40 m of the columns; ii- Updated 3: Model according to Updated 2, but E = 6.13 GPa in the last 0.40 m of the columns.

In Table 3 the experimental and numerical frequencies are presented for each case. Also the E considered in each model is given, and the error between the numerical and the experimental frequency.

Test	Model	Frequency [Hz]	E [GPa]	Error %
Frame without G-FRP, after the first test	Measurement	2.75		
	First Model	3.22	21.02	17
	Updated 1	2.80	15.33	2
Frame without G-FRP, after the fifth test	Measurement	1.5		
	Updated 1	2.80	15.33	87
	Updated 2	1.52	0.66	1

Test	Model	Frequency [Hz]	E [GPa]	Error %
Frame with G-FRP, after the first test	Measurement	2.25		
	Updated 2	1.52	0.66	33
	Updated 3	2.25	6.13	0.2
Frame with G-FRP, after the fifth test	Measurement	2.1		
	Updated 3	2.25	6.13	7
	Updated 4	2.10	4.70	0

Table 3: Updating of E in FEM's models.

4.3 Model updated vs. prototype tested

The acceleration time history obtained in the FEM's model in the frame without G-FRP for dynamic test was compared to the experimental time history, and it is shown in Figure 14. Figure 15 shows the acceleration time history from the set of test of the frame with G-FRP. A good agreement is observed in both cases. In the Table 4 it is showing the RMS errors estimated by the equation 2.

$$Error_{RMS} = \sqrt{\frac{1}{N}\sum_{i=1}^{N}(a_{Model} - a_{Test})^2} \tag{2}$$

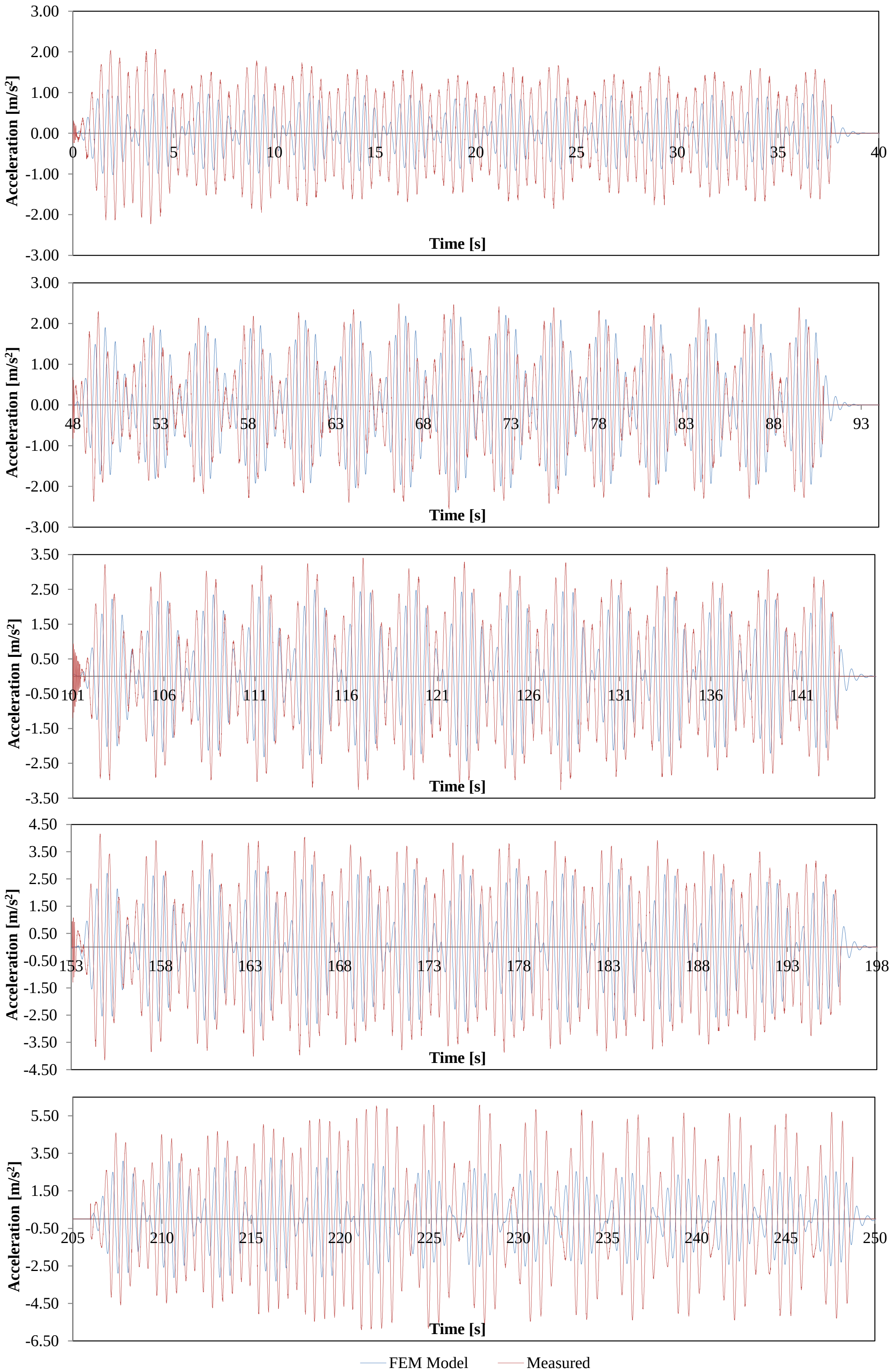
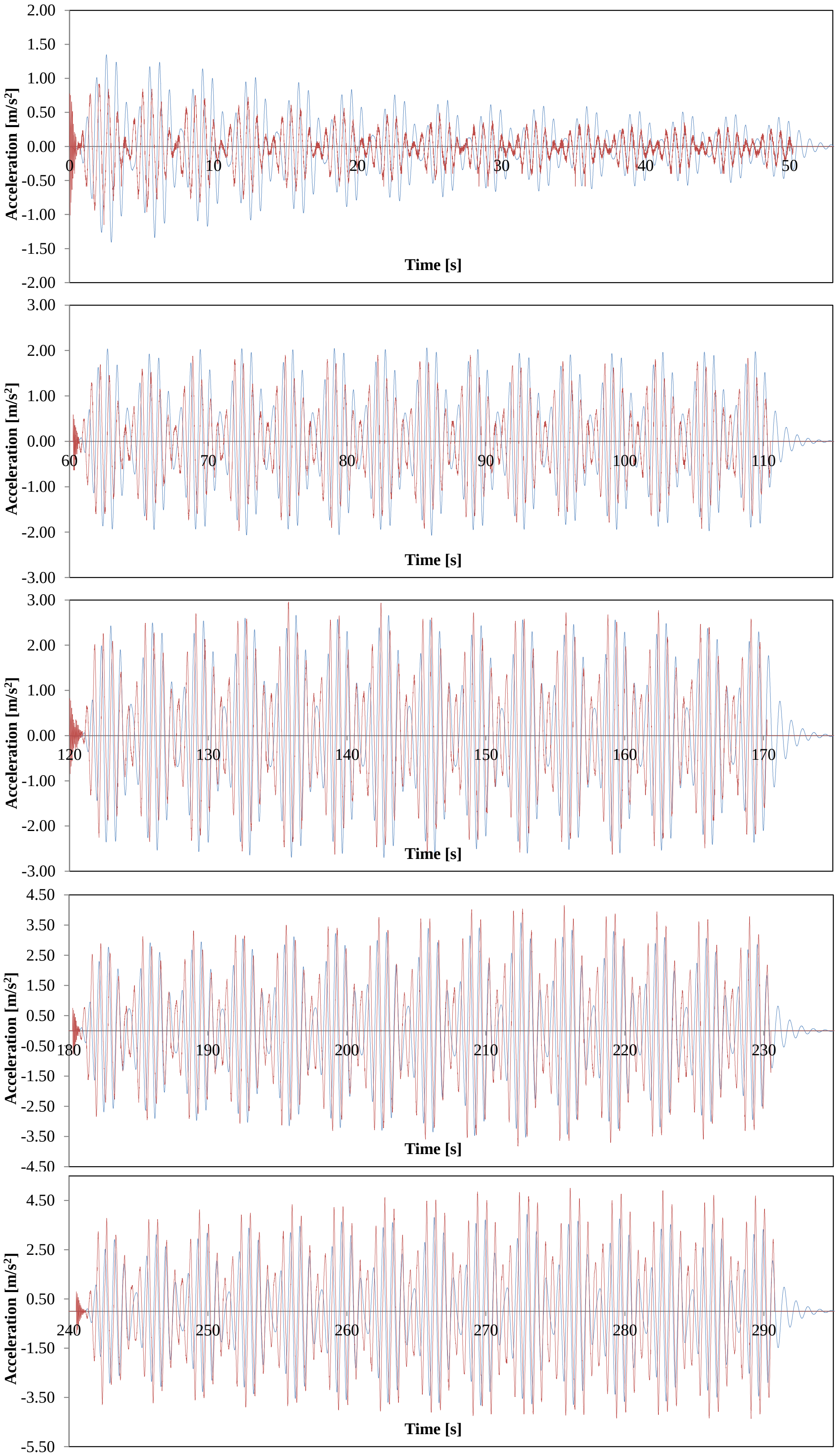


Figure 14: Acceleration time history, numerical vs. experimental measurements for the five tests (Frame without G-FRP).


2.00
1.50
1.00
0.50
0.00
-0.50
-1.00
-1.50
-2.00
Acceleration [m/s²]
0
10
20
30
40
50
Time [s]
3.00
2.00
1.00
0.00
-1.00
-2.00
-3.00
Acceleration [m/s²]
60
70
80
90
100
110
Time [s]
3.00
2.00
1.00
0.00
-1.00
-2.00
-3.00
Acceleration [m/s²]
120
130
140
150
160
170
Time [s]
4.50
3.50
2.50
1.50
0.50
-0.50
-1.50
-2.50
-3.50
-4.50
Acceleration [m/s²]
180
190
200
210
220
230
Time [s]
4.50
2.50
0.50
-1.50
-3.50
-5.50
Acceleration [m/s²]
240
250
260
270
280
290
Time [s]

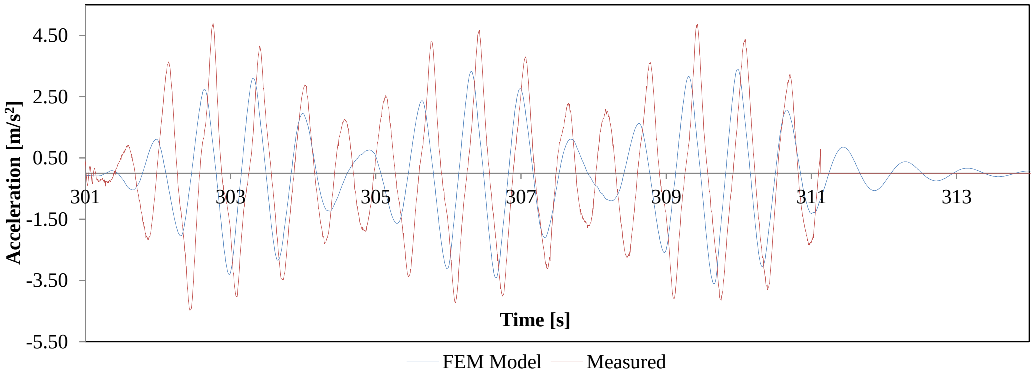


Figure 15: Acceleration time history, numerical vs. experimental measurements for the six tests (Frame with G-FRP).

Test	Accel. [m/s²]	Displ. [m]
1°	1.08	0.004
2°	1.57	0.009
3°	1.92	0.009
4°	2.44	0.012
5°	3.78	0.026

Test	Accel. [m/s²]	Displ. [m]
1°	0.48	0.006
2°	1.05	0.014
3°	1.31	0.017
4°	1.65	0.022
5°	1.96	0.024
6°	1.91	0.022

Table 4: RMS errors for the eleven tests, without G-FRP (left) and with G-FRP (right).

5 CONCLUSIONS

The evaluation of damage through system identification is a good tool for the analysis of a structure as a method of non-destructive test. With system identification it is possible to know the efficiency of a retrofitted applied to a structure.

The problem considered in this paper was the updating of a FEM's model of a reinforced concrete frame, to get good agreement between numerical and experimental responses. A frame with / without G-FRP was studied.

The updating was based on the frequencies measurement by simple system identification techniques, and the modification of the Young's modulus E for the concrete model.

It is possible to get a FEM's model that simulate very well the real structure, but if the structure is much damaged, the basic updating procedure will not be enough, as seen in the last test of each of the two sets (with and without FRP) and also the error increased with the level of damage.

It is necessary to study new procedures to improve the updating FEM's models of structures that have been seriously damaged.

ACKNOWLEDGEMENTS

F. Calderón thanks the support of the National Agency of Science and Technology of Argentina.

REFERENCES

Vélez, W., Gómez, D., Thomson, P. “Finite Element Model Updating” Dyna, Año 76, Nro. 158, pp. 177-189. Medellín, Junio de 2009. ISSN 0012-7353.

Friswell, M.I., Mottershead, J.E. “Model Updating in Structural Dynamics: A Survey”. Journal of Sound and Vibration 167(2), 347-375, 1993.

Ahmadian, H., Mottershead, J.E., Friswell, M.I. “Regularisation Methods for Finite Element Model Updating”. Mechanical Systems and Signal Processing 12(1), 47-64, 1998.

Dunn, S.A. “Issues concerning the Updating of the Finite Element Models from Experimental Data”. NASA Technical Memorandum 109116. Langley Research Center, Hampton, Virginia 1994.

Hemez, F.M., Doebling, S.W. “Review and Assessment of Model Updating for Non-Linear, Transient Dynamics”. Mechanical Systems and Signal Processing 15(1), 45-74 2001.

Datta, B.N. “Finite Element Model Updating, Eigenstructure Assignment and Eigenvalue Embedding Techniques for Vibrating Systems”. Mechanical Systems and Signal Processing 16(1), 83-96, 2002.

Calvi, A. “Uncertainty-Based Loads Analysis for Spacecraft: Finite Element Model Validation and Dynamic Responses”. Computers and Structures 83, 1103-1112, 2005.

Bishop, R. and Gladwell, G. An investigation into the theory of resonance testing. The Royal Society, 1961.

Clough, R. and Penzien, J. Dynamics of structure. United Stated: McGraw-Hill, 1975.

INTI-CIRSOC. CIRSOC 201 – Argentinian Code of Concrete Structures. Argentina, 2005.

INTI-CIRSOC. INPRES-CIRSOC 103 – Part I – Argentinian Code for Seismic Construction. Argentina, 1991.

Chopra A. K. Dynamics of structures. United Stated: Prentice Hall, 1995.

SeismoStruct V. 5.2.2. Software applications for analysis of structures subjected to seismic actions. Italy: SeismoSoft Ltd, 2010.

Palazzo G. Damage Identification in structures thru the changes in the dynamic characteristics. Argentina: National University of Tucuman, 2000.

Palazzo, G, Martin, P. Calderón, F. and Roldan, V. Numerical simulation of a reinforced concrete frame subject to dynamic actions. Argentina: Proc. of MECOM 2012.

Kinemetrics. Strong Motion Analyst, 2004.

Madas P., Advanced Modelling of Composite Frames Subjected to Earthquake Loading. PhD Thesis, Imperial College, University of London, London, UK, 1993.

Mander J.B., Priestley M.J.N., Park R., Theoretical stress-strain model for confined concrete. Journal of Structural Engineering, Vol. 114, No. 8, pp. 1804-1826, 1988.

Martinez-Rueda J.E., Energy Dissipation Devices for Seismic Upgrading of RC Structures. PhD Thesis, Imperial College, University of London, London, UK, 1997.