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Identification of dynamic response parameters of a concrete building during recent earthquakes using structural monitoring

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Abstract

The main building of the Faculty of Engineering at Pontificia Universidad Católica de Valparaíso, emplaced in Valparaíso city at central coastal region of Chile, has been instrumented with a vibration sensor network that consists of three synchronized triaxial strong motion accelerometers. This reinforced concrete building structured with shear walls and frames consist in one underground level and five levels over-ground.

Since 2015, September to 2016, April, it was subjected to 122 sensible earthquakes with magnitude (M_I or M_w) between 4.5 and 8.4. The acceleration records were collected for each earthquake in two orthogonal-horizontal directions and the vertical direction in three different floors of the building: base (-1 level), second floor and fourth floor. Previously, during 2014 the initial conditions of the measure were obtained, before the study of earthquakes occurred, using velocity and acceleration records of micro-tremors: the soil's resonant period and structure's vibration periods were obtained using frequency domain analysis.

Techniques of dynamic system identification based on both the time domain (space-state equations) and the frequency domain (Fourier analysis, EFDD) have been applied and compared. The correlation of the response parameters with some intensity seismic parameters recommended by the literature also was studied. This enabled the assessment of variation of the parameters of dynamic response, such as the vibration periods in relationship to the variation of the seismic intensity parameters. Also, know the history of the dynamic response parameters under analysis before and after each earthquake enabled the study of damage states in this specific structure.

The main results indicate that all the identification used techniques obtain similar estimation for modal vibration periods: these modal vibration periods are not constant during the time and they can be influenced by the seismic intensity although the soil motions have low intensities.

Also was observed that the variation of modal vibration periods between before and after of the used earthquakes (soft earthquakes) is on order of the intrinsic variation of the identification methods.

Keywords: Structural health monitoring; dynamic response parameters; identification of dynamic systems

1.INTRODUCTION

It is well know that for the traditional structural analysis and design, the building's dynamic response parameters are considered constants and independents of the seismic excitation or other environmental conditions. Commonly also are not measured or assessed experimentally one or some dynamic response parameters for to compare them with design values used. Recent research shows that the environmental and/or excitation conditions influence severally the response of the structures [1, 2], including an important uncertainly in the mathematical models that it is normally neglected in normative discussions and do not exist formal guidelines for assessing the consistency between the mathematical model used with the building built.

In consideration of this dichotomy, the past December of 2014 a vibration monitoring network was installed in the Facultad de Ingeniería building (FIN), at the Pontificia Universidad Católica de Valparaíso (PUCV), with the purpose of monitoring its structural health using strong motion and microtremor records. This information is used to identify parameters of dynamic response of the building and its variations over time.

In this work, the study of time variation of dynamic response parameters is developed considering 122 sensible earthquakes with magnitude between 4.5 and 8.4 (M_w or M_l) occurred between September of 2015 and April of 2016 at the central coastal zone of Chile (where the FIN-building is located). Those earthquakes were recorded by the vibration sensor network installed in the FIN-building (base: underground level; second and fourth floor) and they were used in different signal analysis: non-parametric frequency (Peak Peaking method); Enhanced Frequency Domain Decomposition (EFDD) and time domain analysis (space-state equations, using N4SID algorithm). Also, for each earthquake record some seismic intensity parameters commonly used in the literature as peak ground acceleration and I_a [3] were obtained and used to study the variation of dynamic response parameters of FIN-building.

2.METHODOLOGY AND RESULTS

2.1. Building and vibration network information.

FIN-building have five stories and one basement floor. Its structure consists in reinforced concrete shear walls and reinforced concrete frames, with a lightweight shed of steel in the roof (fifth floor). The purpose of this building is educational and it is located in the central coastal city of Valparaíso, Chile. Fig. 1 contains a frontal view of the building and its emplacement. Further information in [4].

Fig. 1 – FIN-building: Frontal view and emplacement.

The vibration sensor network consists in three tri-axial force balanced accelerometers model SARA-SL06 installed in three different levels of the building: the basement (-1 level), second floor and fourth floor. Their location in each floor and their identification code are indicated in Fig. 2. The longitudinal direction of the building and accelerometers are aligned with the North-South direction, while the transversal direction of the building and accelerometers are aligned with the East-West direction.

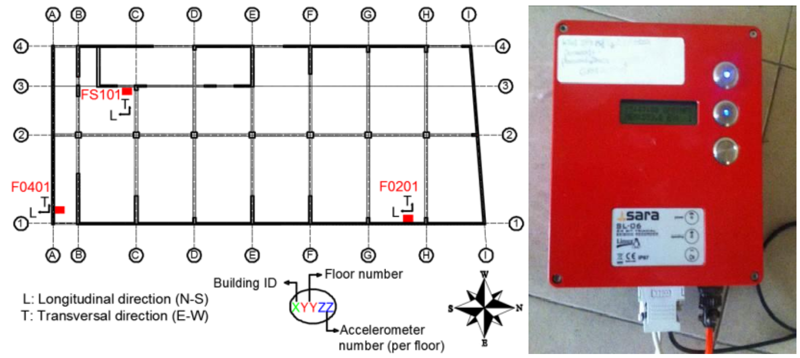

Fig. 2 – Accelerometers location and sensor

All the accelerometers are networked to obtain synchronized and simultaneous seismic data with sampling ratio of 200 Hz. Each one has its own battery system for autonomous operation in the case of blackouts, and they are connected to an external GPS for UTC time synchronization. Five days of continuous data is recorded before its permanently erased. For the strong motion acquisition, an STA/LTA trigger was configured using the initial recommendations of Trncoczy [5].

2.2. Previous measurements and characterization of soil and building

Previous identification analysis was made by Orrego in 2014 [4], before the operation of the seismic sensor network started. Using a Tromino ® Engy 3G sensor (by Micromed/Moho), from analysis of microtremor records with duration of 16 min, the soil resonant period was identified using the Nakamura's technique [6], while, from triaxial microtremor records with duration of 30 min in each floor (2nd and 4th), obtained by the accelerometer network, the building's vibration modal periods in both orthogonal-horizontal direction (N-S and E-W) were estimated using the Enhanced Frequency Domain Decomposition method (EFDD) proposed by Brincker, Zhang and Andersen [7]. Table 1 contains the modal periods of the FIN-building, while Fig. 3 shows the soil resonant period identification.

Table 1 – Modal period identification from the previous analysis (EFDD method)

Mode	Modal period left limit (s)	Modal period value (s)	Modal period right limit (s)
1	0.2962	0.3005	0.3055
2	0.1886	0.2072	0.2308
3	0.1254	0.1297	0.1339
4	0.0962	0.0989	0.1017

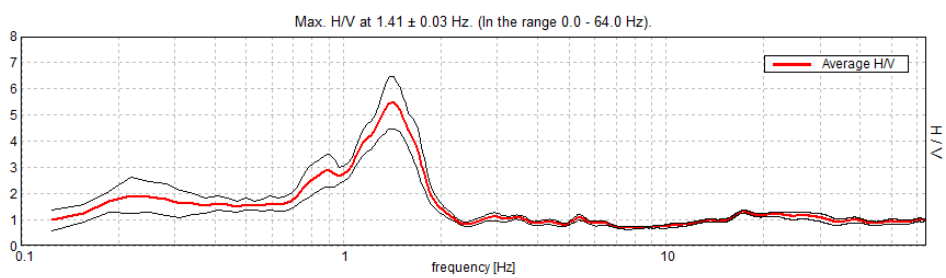

Fig. 3 – Soil resonant period identification

2.3. Strong motion database

The earthquakes database used in the analysis comprehends strong motion records from 122 earthquakes occurred between September, 16th 2015 and Abril, 25th 2016 with M_I or M_w reported between 4.5 and 8.4, and approximate epicenter distances reported between 537.6 km and 49.2 km. The first earthquake considered in the database corresponds to September, 16th 2015 Illapel earthquake (M_w8.4) that occurs near the coastal cities of Coquimbo and La Serena, Chile. The identification number for each event is assigned as they occur. Continuous data of that day was saved to identify the initial conditions of the records. The date, magnitude (M_I or M_w) and epicenter coordinates were obtained from Centro Sismológico Nacional (information available on www.sismologia.cl).

Some seismic intensity parameters studied by Riddell [3] are obtained for each record in each direction (N-S and E-W): peak ground acceleration (PGA) and Riddell-García intensity index (I_a) [8] were calculated for each earthquake record. These indices were selected due to their good correlation with the response structures with short vibration periods, as shown in Riddell [3].

Fig. 4 contains the mentioned intensity values. For the calculus of PGA and I_a, an 8th Butterworth band-pass filter with cut-frequencies between 0.1 Hz and 30 Hz had been applied to the data.

PGA between 0.04% and 5.41% of g were observed. In other hand, the I_a values achieved oscillate between the 1.36 cm/s$^{5/3}$ and 217.73 cm/s$^{5/3}$.

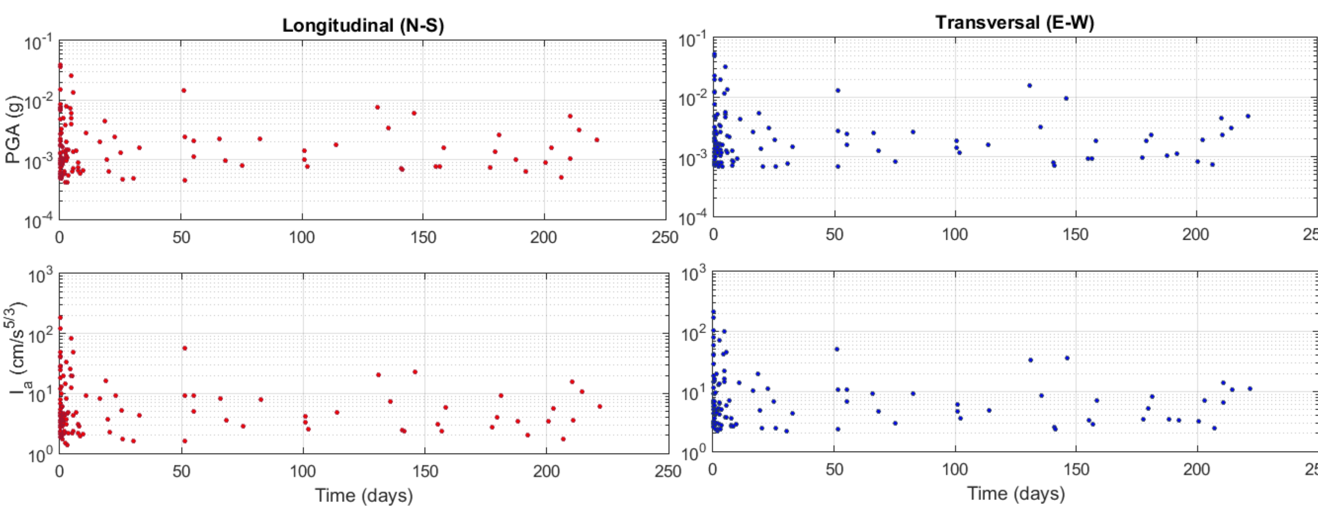

Fig. 4 – Seismic intensity indices of earthquake database

2.4. Periods estimation

The techniques used for structural period identification in this work are three: the Space-State method (N4SID), used in system identifications (as can be seen in Ljung [9]), the Peak Picking Method (as described in Bendat and Piersol [10]), and the EFDD method [7].

2.4.1. Space-state method (SSM)

This technique [9], which works in the time domain, is a Single Input-Single Output (SISO) stochastic method. The parametric identification works iteratively with different model orders to discriminate the physical modes of the structure, based on stability analysis (stability diagrams). The model used in this work start with the 4th order and it grows in pairs till the 24th order. Due to there are two outputs (2nd and 4th floor records), two different models has been identified with in theory the same information.

The automatization of this analysis was made with N4SID algorithm. The criteria for the stability diagrams interpretation was as follow: first, the validation of the dynamic response parameters was tested; second, the dynamic response parameters were grouped. Table 2 described the different criteria applied to the period and damping ratio values. Fig. 5 contains the modal periods identified using the State-space method.

Table 2 - Validation and Grouping criteria

Validation Criteria	Grouping Criteria
Damping ratio less than 20%	Frequencies with differences less than 15% of the lower value are considered part of the same mode
Period analysis range between 0.07 s and 0.5 s	Minimum 5 frequency values per group. Frequency estimation bandwidth: 40% of first frequency
Period and damping ratio obtained from the state matrix eigenvalues in complex and conjugate pairs	The representative modal frequency is the average value of the group

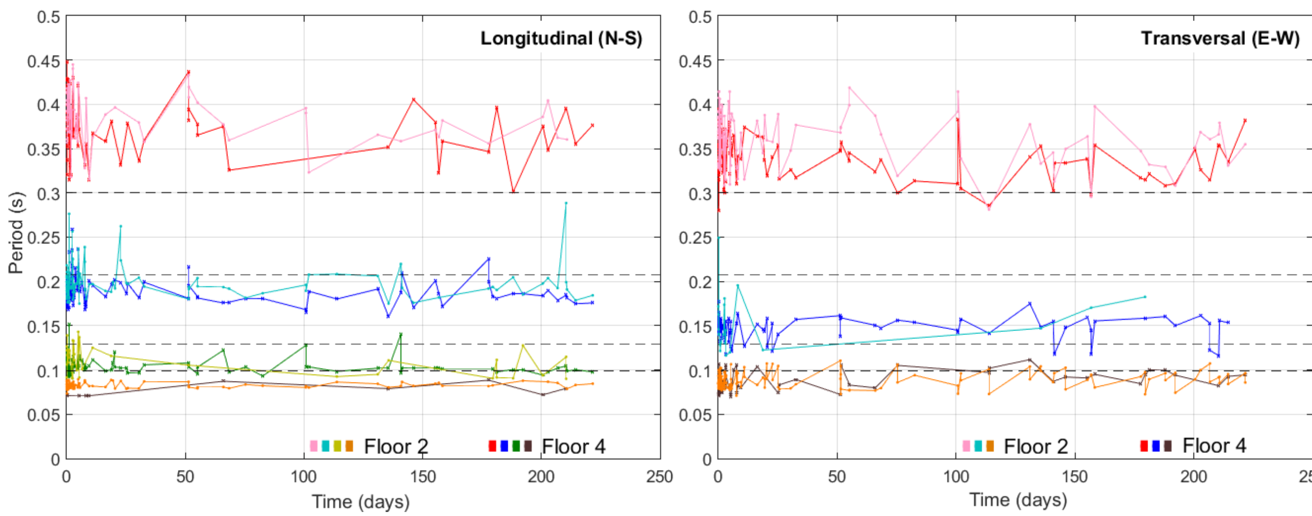

Fig. 5 – Period identification by SSM

2.4.2. Peak picking method (PPM)

This non-parametric method, which works in the frequency domain with only output signal [10], uses the Fast Fourier Transform on the seismic recorded signal to compute and obtain the power spectral density function and in it identify each period associated with each local peak as a structure's vibration period. This only works properly if the identify periods are well separated of each other.

The period range under analysis (0.07 s to 0.5 s) was the only consideration used with this technique. Fig. 6 contains the periods identified using the peak picking method.

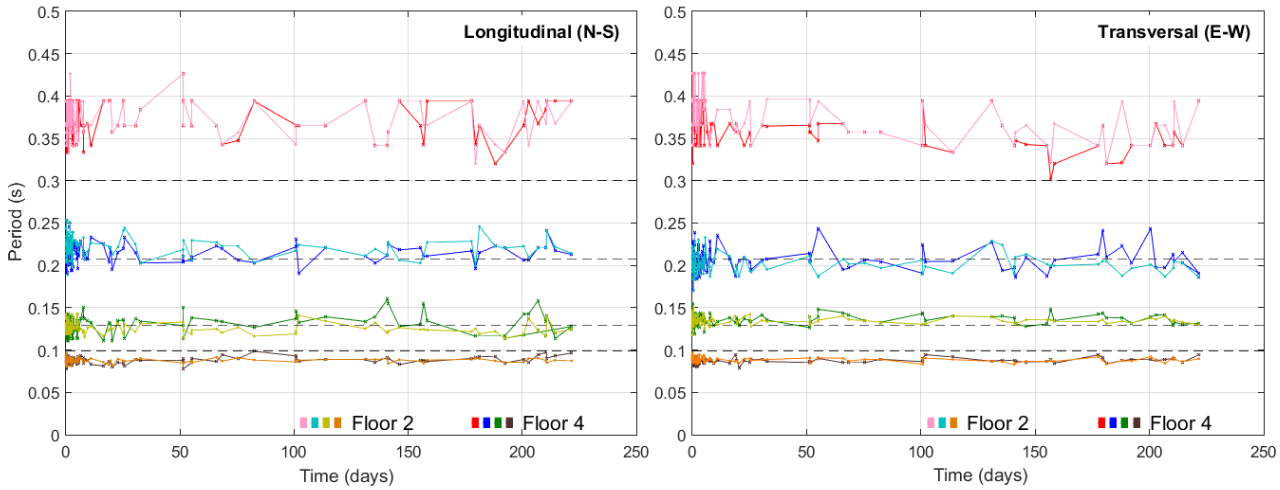

Fig. 6 - Periods identification by PPM

2.4.3. Enhanced frequency domain decomposition (EFDD)

This technique, which works in the frequency domain using only output signals [7], is a variation of the classic Peak Picking technique. By using the singular value decomposition of the spectral matrix, a set of auto power spectral density functions is obtained. Each of this function corresponds to a single degree of freedom system.

The first singular function for every earthquake of the database was stored and analyzed. Table 3 described the period identification criteria. Fig. 7 contains the periods identified using the EFDD method.

Table 3 – EFDD period selection criteria

Period analysis in the range 0.07 s and 0.5 s (approximately)
Normalized Singular Values functions for peak selection
95% for coherence using the Modal Assurance Criterion (MAC)
Manual selection for the period estimation bandwidth for each mode

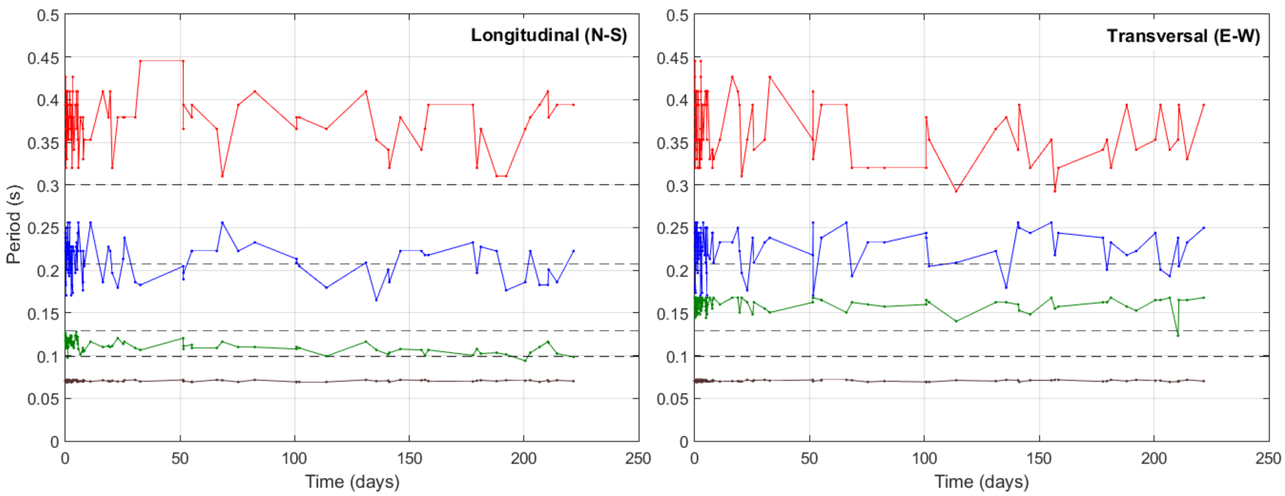

Fig. 7 – Period identification by EFDD method

3. ANALYSIS AND DISCUSSION

The average ($\bar{T}$), standard deviation (S_T) and coefficient of variation (COV) were calculated for each set of modal periods identified with the techniques described in the previous section (see Table 4). As can be seen, the three techniques used obtain very similar results in terms of $\bar{T}$, S_T and COV for all identified vibration modes and, in general for the modes with higher average periods, they are higher than the periods identified using the EFDD method with microtremor records (see Table 1). Due to the microtremors can be considered as earthquakes with a very low intensity, these results suggest that the intensity of soil motion has a non-negligible influence on the mains vibrations period of the structure. In the same way, in Figs. 5, 6 and 7, the black dash line indicate the reference period obtained by Orrego [4] (see Table 1) where the two modes with higher periods are generally higher this dash lines.

Table 4 - Statistical values for each set of modal periods

Mode	Technique	Floor	Longitudinal (N-S)			Transversal (E-W)		
			$\bar{T}$ (s)	S_T (s)	COV (%)	$\bar{T}$ (s)	S_T (s)	COV (%)
1	SSM	2	0.3850	0.0284	7.39	0.3594	0.0292	8.12
		4	0.3754	0.0316	8.42	0.3428	0.0239	6.98
	PPM	2	0.3723	0.0199	5.34	0.3788	0.0259	6.84
		4	0.3711	0.0202	5.44	0.3664	0.0254	6.93
	EFDD	-	0.3699	0.0259	7.00	0.3595	0.0360	10.01
2	SSM	2	0.1987	0.0183	9.21	-	-	-
		4	0.1894	0.0149	7.87	-	-	-
	PPM	2	0.2215	0.0119	5.37	0.2026	0.0106	5.23
		4	0.2151	0.0118	5.49	0.2071	0.0138	6.64
	EFDD	-	0.2092	0.0220	10.52	0.2257	0.0263	11.65
3	SSM	2	0.1110	0.0157	14.11	0.1531	0.0320	20.92
		4	0.1055	0.0084	7.93	0.1482	0.0117	7.90
	PPM	2	0.1265	0.0066	5.18	0.1349	0.0047	3.46
		4	0.1293	0.0110	8.47	0.1356	0.0057	4.22
	EFDD	-	0.1100	0.0067	6.09	0.1595	0.0077	4.83
4	SSM	2	0.0822	0.0031	3.80	0.0881	0.0084	9.57
		4	0.0753	0.0067	8.85	0.0873	0.0111	12.77
	PPM	2	0.0875	0.0027	3.05	0.0888	0.0026	2.95
		4	0.0875	0.0038	4.31	0.0875	0.0030	3.42
	EFDD	-	0.0704	0.0008	1.14	0.0705	0.0008	1.13

Other interesting aspect of the results is shown in Figs. 8 and 9, where the influence of seismic intensity (PGA and I_a) on the modal period, is presented. Again, is possible to see that the periods' estimation using microtremor records analysis understates the period of the modes with higher values (modes 1 and 2 in Table 4). Also, can be seen that while the seismic intensity increase (both PGA and I_a), the estimated higher period increases too. This is not very clear for the others vibration modes.

On the other hand, the analysis of pre-event and post-event records has been developed. For each earthquake recorded, when the STA/LTA trigger was activated, a pre-event record of 60 s was included in the acceleration record. In the same way, when the STA/LTA trigger was turned off a post-event record of 60 s, also was included in the total earthquake acceleration record. Then, the modal vibration periods of the structure during the pre and post-event has been estimated with the three techniques previously indicated. As SSM and PPM generate practically the same information from the analysis of 2nd floor's records and of 4th floor's records, this part of analysis and discussion has been limited to the results obtained from the 4th floor records.

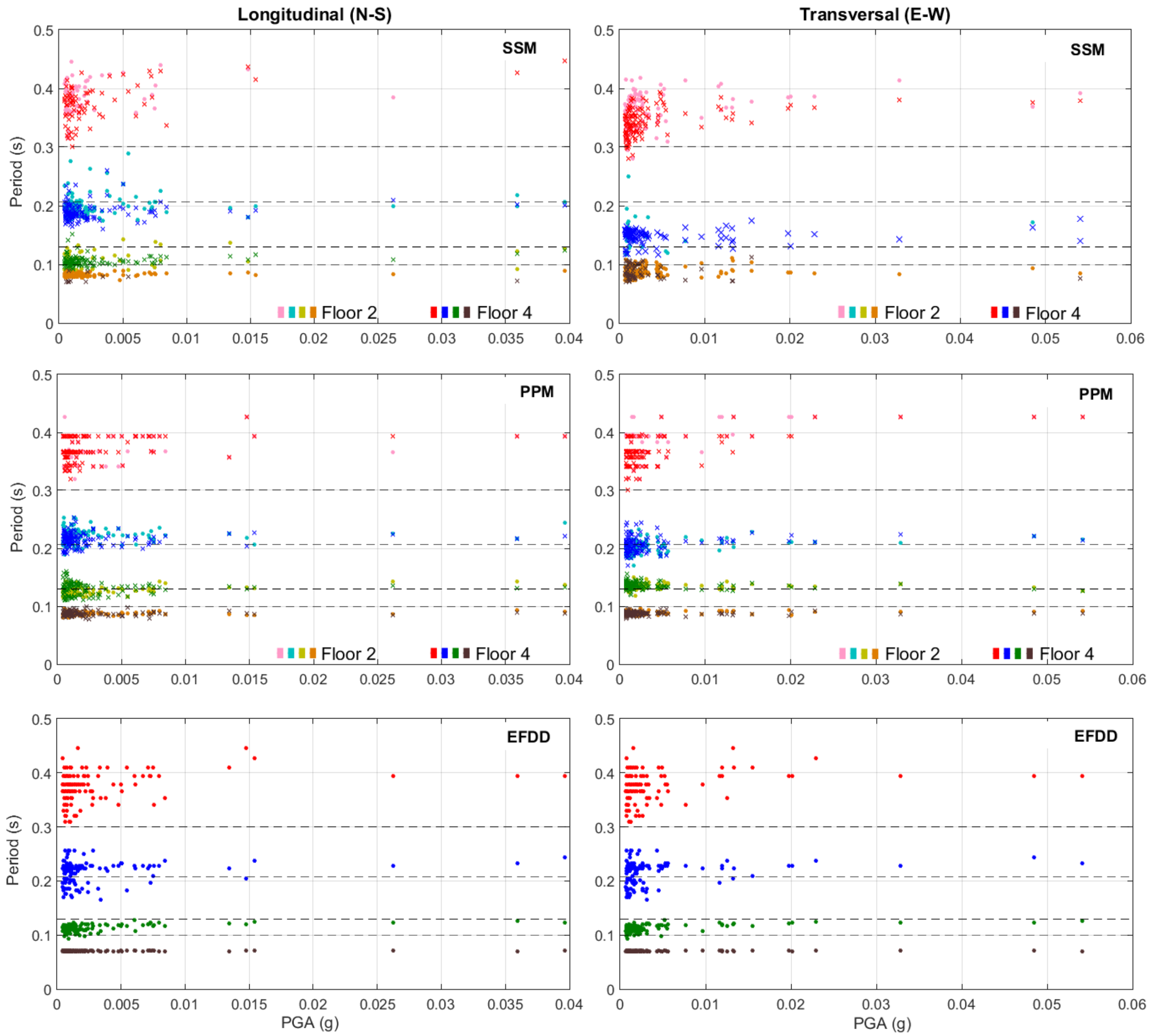

Fig. 8 – Modal periods vs seismic intensity index (PGA)

The Figs. 10 and 11 show the T_f/T_i ratio versus the seismic intensity index (PGA and I_a respectively), where T_i is the modal vibration period of the structure during the pre-event and T_f corresponds to the modal vibration period of building during the post-event time. In these Figs, can be seen that the T_f/T_i ratios oscillate around 1.0 for all modal vibration periods, for all identification techniques used and for the observed range of the both seismic intensity indexes. In the Table 5, the T_f/T_i ratio for each observed vibration mode are introduced: it is possible to see that the T_f/T_i ratios indicate that before and after of each earthquake, the variation of the modal vibration periods have an inherent variation close to S_T (note that the T_f/T_i ratio is close 1+COV, see Table 4), then, this variation is not attributable to damages or other permanent effects and it is more probable that this variation corresponds to the intrinsic dispersion of results of applied methods. This sentence is consistent with in situ observations, which indicate that no damages were observed due to the recent earthquakes and that the building remained in elastic range during these events.

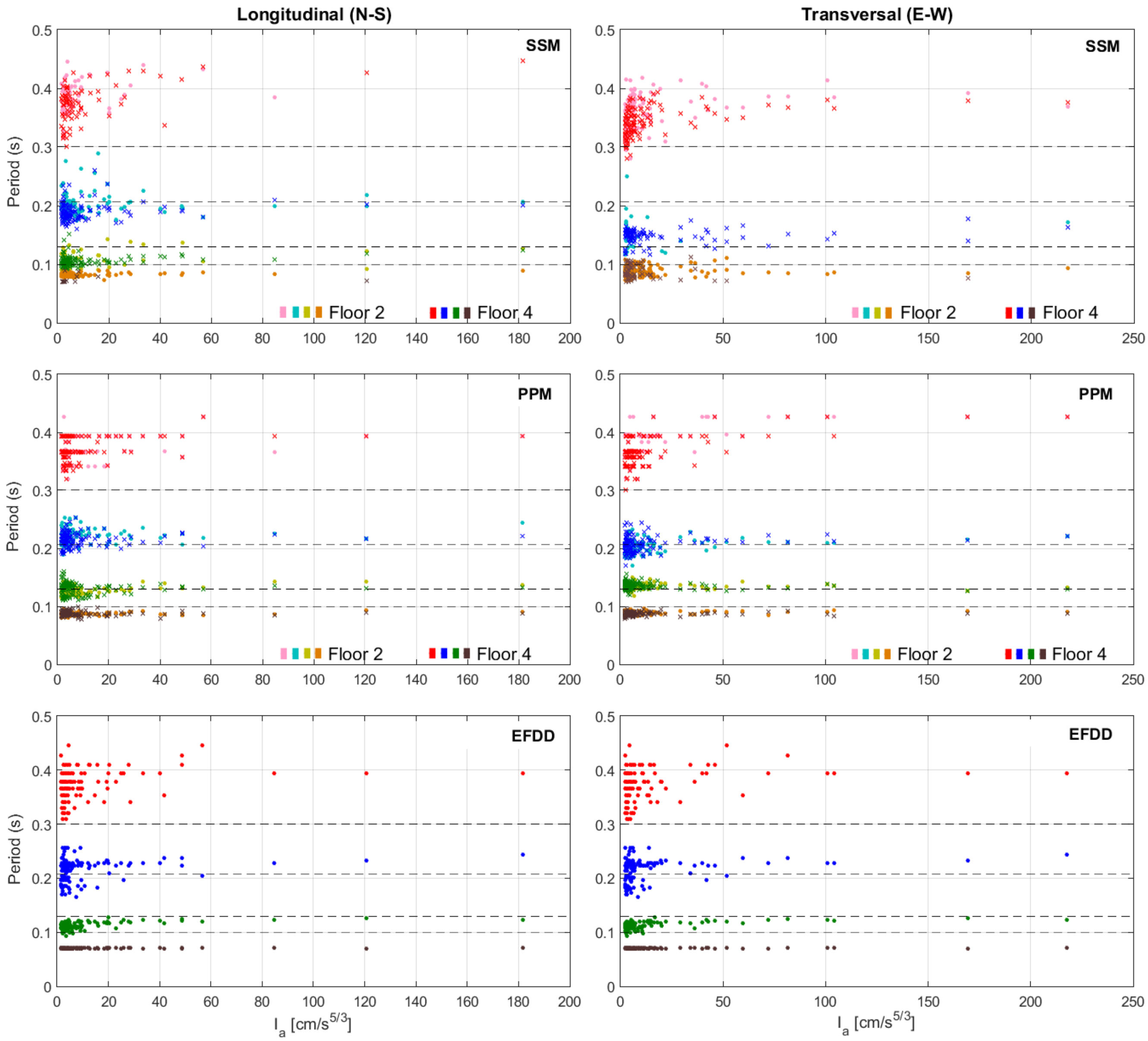

Fig. 9 – Modal periods vs seismic intensity index (I_a)

Table 5 - T_f/T_i ratio for vibration modes by identification technique

Identification technique	Longitudinal direction				Transversal direction			
	Mode 1	Mode 2	Mode 3	Mode 4	Mode 1	Mode 2	Mode 3	Mode 4
SSM	1.0856	1.0295	0.9713	1.0662	1.0325	0.9537	-	1.0302
PPM	1.0561	0.9967	0.9723	1.0025	1.0305	0.9559	0.9922	0.9978
EFDD	0.9289	0.9504	0.9673	0.9744	0.9440	0.9549	0.8894	0.9752

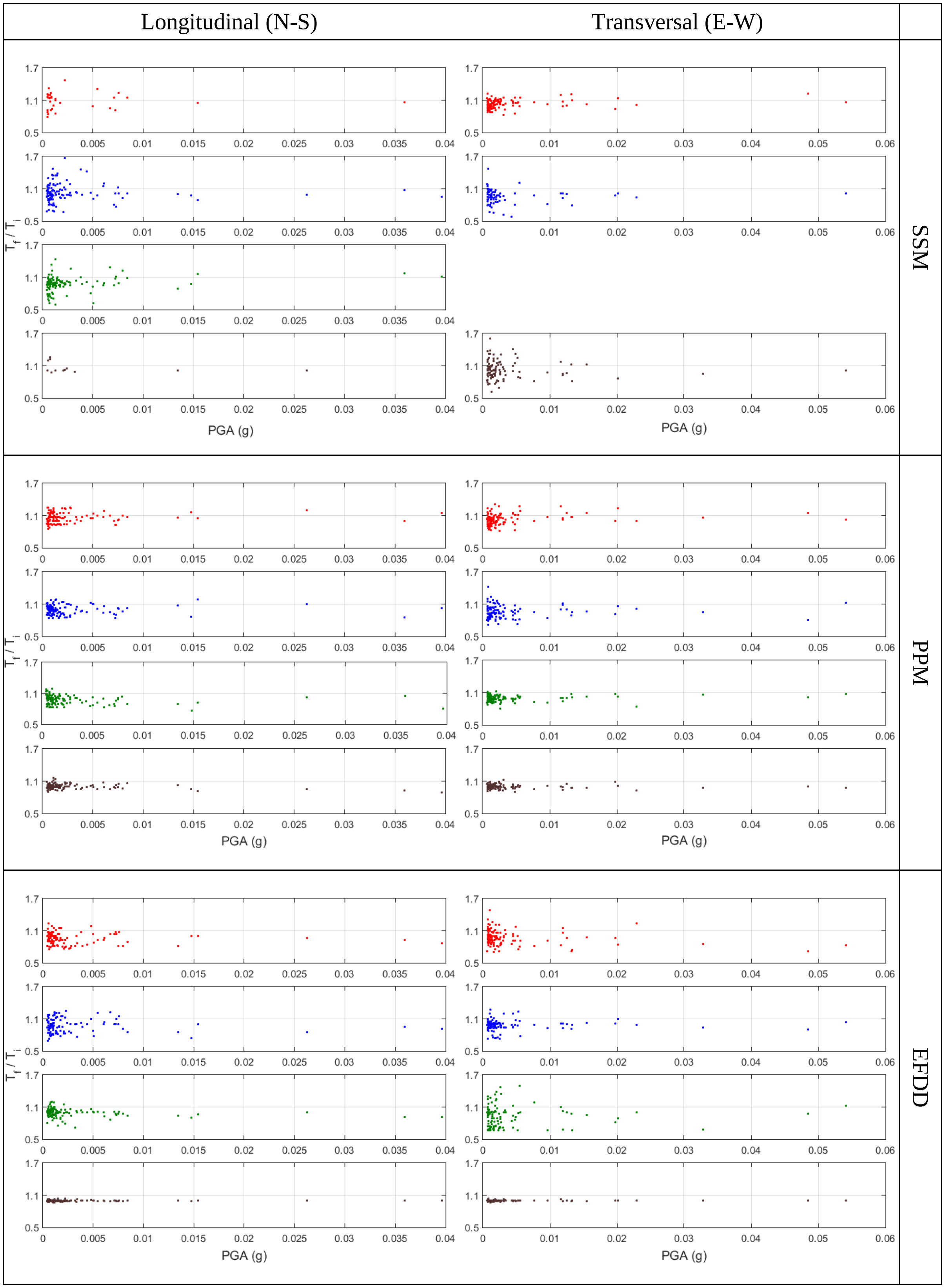

Fig. 10 – T_f/T_i ratios by method vs PGA

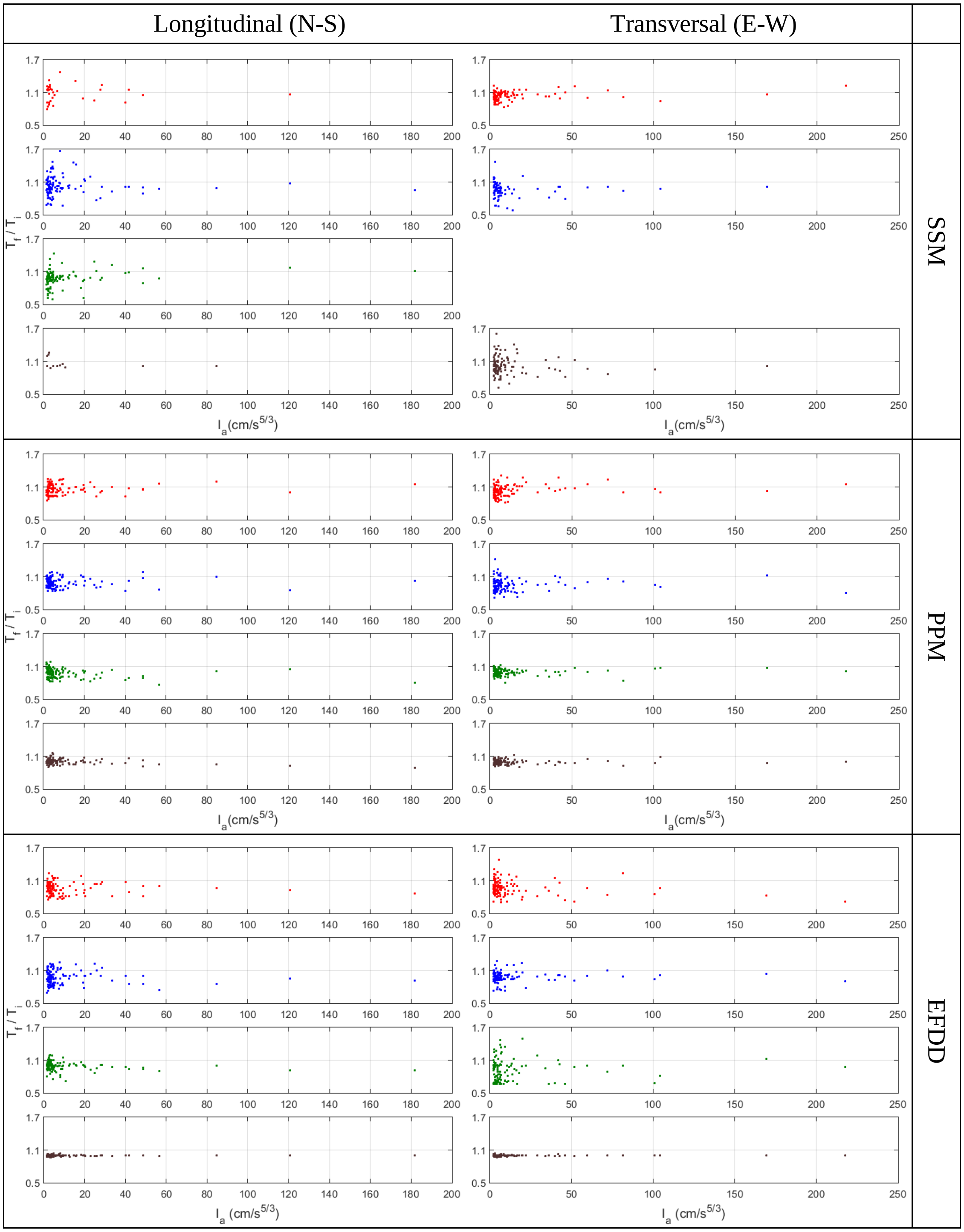

Fig. 11 – T_f/T_i ratios by method vs I_a

4.CONCLUSIONS

This study has presented the application of three methodologies for system identification on a reinforcement concrete building subjected to 122 earthquakes of different magnitudes and intensities. The applied techniques permitted to study the time-variation and the influence of the seismic intensity on modal vibration periods. Before/after earthquake modal vibration periods were studied too.

Results indicate that all identification techniques obtain similar estimation for modal vibration periods: these modal vibration periods are not constant during the time, presenting a variation smaller than +/-15% referred to the average modal period.

The seismic intensity has non-negligible influence on the higher modal vibration period although the soil motions have low intensities. While the motion intensity increases, the higher modal vibration period, increases too. This influence is no clear for the other vibration modes with smaller period values.

For all vibration modes identified with the different applied techniques, the variation between before and after of each earthquake had the same order of the intrinsic variation of each method, then is not possible to attribute any variation of vibration period to damages. In fact, the studied building doesn't have damages even if a slight variation is observed in the higher period of vibration

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