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STRUCTURAL HEALTH MONITORING OF AN INSTRUMENTED BUILDING IN MEXICO WITH ACCELEROMETERS AND GPS SENSORS

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ABSTRACT

This paper presents the most significant results of the structural response of an instrumented building in Mexico during eight seismic events recorded between 2011 and 2013. A criterion for assessment of the structural health is proposed and the advantages of an automatic post-event structural warning system are discussed. The estimation of structural condition is based on five indicators: two related to intensity (peak ground acceleration and Arias intensity) and three related to structural response (interstorey drift ratio, seismic coefficient and percentage change in the fundamental frequencies corresponding to the horizontal components). When the structure is subjected to an earthquake, the records are analysed by the warning system and a report is generated automatically. Changes in structural features can be detected and compared to their initial values to establish their state of damage

KEYWORDS

Structural health monitoring, instrumented building, structural warning, structural response.

INTRODUCTION

The studied building is appreciated as representative of the architecture of Mexico City in the 1960's (Figure 1), the north side is adjacent to the archaeological site of Tlatelolco. The building consists of a 22-storey tower and three low-rise structures. The four components of the building share a common basement. Since 1964, when the building was inaugurated, the building exhibited several problems. The tower had differential settlement and tilting, and uplifting of the low-rise structures.

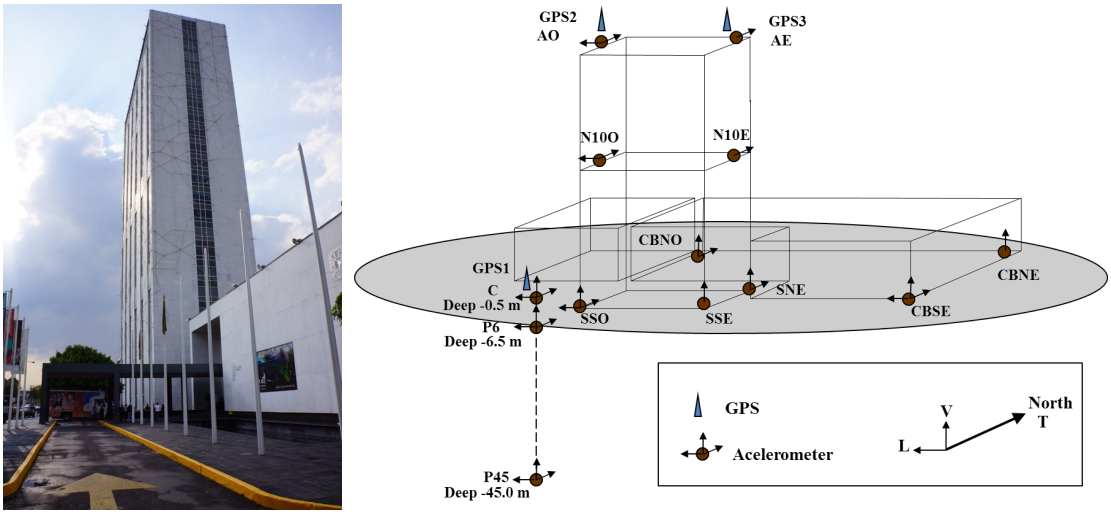


Figure 1. Overview of the building and instrumentation

In order to control these problems, there have been four underpinning in different years and the reinforcement of the tower structure between 2008 and 2009. The last restoration consisted in the stiffening of the tower with steel diagonals in the transverse direction and the retrofitting of the coupling beams by wrapping with carbon-fiber reinforced polymer mesh. The foundation of the tower was expanded in the south side, 30 control

piles were added, and transversal reinforced concrete walls were built between the foundation tower and low-rise structures. The rehabilitation and reinforcement were completed in 2009. Geotechnical studies conducted show that the regional subsidence in the study area is 7 cm per year (Lopez and Avila 2007; Rodríguez *et al.* 2009). In recent years, the observed increase in the tilting of the tower has been small.

The purpose of this paper is to present the analysis of the data obtained from the continuous monitoring of the building response after the rehabilitation with a monitoring system consisting of accelerometers and GPS, and discuss the performance an automatic structural warning system implemented in the building.

MONITORING SYSTEM

The seismic instrumentation in the building, operating since December 2008, consists of a network of 28 high-resolution uniaxial force balance accelerometers (Kinemetrics) located in different points of the building and the surrounding ground, as show in Figure 1. The resolution used to monitor the structural response is adequate for recording a broad range of excitation levels (such as ambient vibration and high-intensity earthquakes). In May 2009, a GPS system (Leica) was installed; this system is composed of a fixed base station (GPS1) and two antennas (GPS2 and GPS3) on the roof of building to monitor the verticality. Starting in August 2011, the GPS system has been operating in parallel with a logging system that records signals from accelerometers, enabled to directly measure the relative displacement at the roof with respect to its reference base station.

With this instrumentation, eight seismic events have been recorded between February 2011 and June 2013, and there have been also several ambient vibration tests being conducted to determine basic dynamic properties (Table 1). Building inspections showed some damage from the earthquakes of December 12, 2011 (denoted as event 11-3) and March 20, 2012 (denoted as event 12-1). For event 11-3, only small cracks in the finished plaster were observed in some masonry walls. Event 12-1 has been the most intense earthquake recorded up to date, and some cracks and detachments in the concrete and masonry walls coverings there were observed. Damage in the mechanism of one of the elevators was also noted. The maximum accelerations of the earthquake of March 20, 2012 were 163 and 29 cm/s² on the roof and the ground adjacent to the building, respectively. The other events did not produce visible damage to the building. The characteristics of the different seismic events recorded, corresponding peak ground acceleration (PGA) and the Arias Intensity (I_{Arias}) of the horizontal components at the ground (Arias 1970) are shown in Table 1.

Table 1. Characteristics of earthquakes recorded in the building

Event	Date	Local time	M_W	Epicentral distance, in km	I_{Arias}, in cm/s	PGA, in cm/s^2
11-1	25/02/2011	07:07:27	6	430	0.02	0.9
11-2	07/04/2011	08:11:22	6.7	539	0.08	1.6
11-3	10/12/2011	07:47:25	6.5	214	2.85	16.4
11-4	16/12/2011	07:02:23	4.5	364	<0.01	0.4
12-1	20/03/2012	12:02:50	7.4	358	15.04	29.2
12-2	02/04/2012	12:36:42	6	356	0.41	5.4
12-3	11/04/2012	17:55:10	6.4	475	0.42	5.0
13-1	21/04/2013	20:16:34	5.8	392	0.57	6.1

It is noted that the verticality obtained with the GPS system is consistent with data obtained from studies using electronic surveying stations. According to those measurements, the values are practically stable and it is expected that the verticality monitoring in future years confirms it.

Structural Warning System

To expedite the analysis and processing of seismic records, an automatic structural warning was implemented in the building (Murià-Vila *et al*., 2010a and 2010b). After the structure is subjected to an earthquake, the records obtained are analyzed and a report is generated automatically in minutes. The report contains some records and their spectral, the values of the indicators, and an estimate of the possible state of damage. Changes in structural features can be detected and compared to their initial values to establish their state of damage. Subsequently, this

report is reviewed for ratification or possible correction. This provides evidence when taking decisions about functionality and safety of a structure.

The structural warning system implemented in the building is based on five indicators: peak ground acceleration (PGA) of the horizontal components, Arias Intensity (I_{Arias}) of the horizontal components of the ground, average interstory drift ratio (SD), variation of fundamental frequencies (VF), seismic coefficient (C_s) estimated from the accelerations recorded in the structure. The selection of the indicators was supported on the fact that they can be calculated with an automatic procedure, based on the information derived from the instrumentation of the building.

SD is estimated by dividing the relative displacement recorded with the GPS by the distance between the GPS unit at the roof and the reference base station.

The VF is calculated with respect to the reference values obtained from an earthquake of small intensity or ambient vibrations tests. This requires determining the intense phase of the records, as well as the initial and final phases of low intensity, in addition to identifying the frequencies of vibration and defining the reference value. The VF of the intense and final phase is calculated with respect to the initial phase and also to the reference values. For low-intensity earthquakes the VF is calculated considering the frequency identified with full signal and the reference. For moderate and high intensity earthquakes, and in events where the initial phase cannot properly be identified, the frequency is determined using the final phase of the previous event. The warning system reports the value of VF determined at the final phase of low intensity with respect to the established reference values.

The C_s that is generated in the structure during a seismic event is derived from the accelerations registered at each instrumented story and their correspondent masses. From these, inertia forces are obtained at these stories and, by interpolation, forces are calculated at other levels. This is based on the Newton interpolation method and produces very good results, as long as three stories (roof, half height and base) are instrumented at least. From these, shear forces acting at the base of the building are obtained as the sum of the forces on each floor and then are divided by the total weight of the structure to calculate C_s, which then is compared with the design value.

For each indicator a scale of three levels of possible damage is proposed: light damage (level 1), intermediate (level 2) and severe damage (level 3).

The three levels of possible damage are defined for each indicator by setting two thresholds: one from level 1 to 2 and one from level 2 to 3. These thresholds are a function of the building characteristics (structure and non-structural elements), characteristics of the site where the building is located, and its structural condition when monitoring started. In the initial stages of monitoring in a building the indicators may indicate different levels of damage. Experience obtained from studies on the seismic response of four instrumented buildings in Mexico, with 7 to 20 years of service life (Murià-Vila 2007), has made possible to consider various scenarios, and establish reference values and thresholds for each level of damage. Using these findings (Murià-Vila *et al*. 2010a and 2010b), it was proposed to use the threshold values shown in Table 2 for the monitoring of the structure studied.

Table 2. Threshold values selected for the building

Level	A_{max}, in Gal	I_{Arias}, in cm/s	Cs_{max}	SD_{max}, in %	VF_{max}, in %
1	< 15	< 5	< 0.03	< 0.12	< 7
2	15 – 130	5 -140	0.03 - 0.16	0.12 - 0.60	7 - 25
3	> 130	> 140	> 0.16	> 0.60	> 25

The software framework for automatic structural warning proposed in this study was developed by Aldama (2009). The system has the option to update or modify the reference values, and the level of state of damage, depending on the analyzed information of the events that were recorded and the behavior that is observed during the inspection of the structure. All the information generated by the structural warning is issued as follows: acceleration histories at roof and ground level; table of the maximum values of accelerations, velocities, displacements, SD, and I_{Arias}; a comparative table of maximum values of previous events; identification of vibration frequencies and their variations regarding the reference values; and a comparative chart with previous

events; Fourier spectra and spectral ratios (roof-base and roof-ground), as well. Once the structural warning has been triggered, the complete information is reviewed to confirm or modify the report issued automatically.

To set the thresholds, the following information is needed: type of structure, verticality and vibration frequencies of the building, type of foundation and soil, rehabilitation and age of construction, as well as an estimate of their structural condition. Finally, the structural condition of the building is established by weighting the five indicators, based on four colors:

1. Green, when VF,SD, Amax, I_{arias} and C_s correspond to level 1
2. Yellow, when VF or SD, or two of the remaining indicators correspond to level 2
3. Orange (building inspection) may be determined when VF or SD is level 3 and at least one indicator A_{max}, I_{arias} or C_s is Level 2, or VF is Level 2 and one of the other is level 3. In both cases, SD due to uncertainties is not considered in their estimation in case of the GPS sensor failed. In this state, it is urgent to inspect the building in order to exclude the case of a Red State
4. Red, if the inspection suggests to evacuate the building.

The three levels of damage derived from the indicators correlate with light, intermediate and severe damage, as follows.

A. Light or tolerable damage includes the possible presence of cracks in non-structural elements, although some cracks may appear in structural elements such as masonry walls. These damages represent no risk to the occupants. There might be a slight decrease in the frequency of vibration of the building for losses attributable to rearrangements stiffness of nonstructural elements. SD cannot be exceed although such SD may produce some damage to nonstructural and structural elements, but they are of equal or lesser magnitude than those produced by the regular use of the building.

B. Intermediate damage is evident because the presence of cracks in structural and nonstructural elements. Detachment may occur and ceilings may fall. SD may exceed acceptable levels in non-structural and structural elements. The vibration frequency of the building will decrease because the loss of stiffness of the building. This level of damage generally represents no risk to occupants, but eventually it may be necessary to close some areas of the building to carry out repair work. Requires visual inspection of the building by a structural engineer to determine its structural condition and if evidence of poor behavior is observed, it may require a corrective intervention.

C. Severe damage is evidenced by the presence of cracks in structural and nonstructural elements, and a large loss of stiffness. In general, C_s and SD are close to or exceed the allowable building standards. The building requires an urgent visual inspection by a structural engineer to determine its possible evacuation.

ANALYSIS OF EARTHQUAKE RECORDS

Table 3 summarizes the peak accelerations and displacements of the earthquake records of roof level. In all events the displacements computed by integrating the acceleration records and measured with the GPS system are very similar. The maximum accelerations and relative displacements in the L direction were 176.3 cm/s^2 and 16.5 cm, and in T direction 163.0 cm/s^2 and 14.6 cm during the event 12-1. Figure 2 shows the time-histories acceleration at the NE corner roof of the building during the 11-3, 12-1 and 13-1 events. Similarly for those events, in the Figure 3, the histories of relative displacement measured with GPS units in the roof with respect to its reference base in ground floor, and those computed from the accelerograms are compared. These displacement histories show that the GPS data recorded during smaller and moderate events have a good performance except for event 12-1.

Note that displacements recorded by the GPS are only available after event 11-2. The displacement amplitudes for the low-intensity seismic event 11-4 were slightly higher than those produced by ambient vibration tests. During event 12-1, no recordings from GPS unit at the roof SW corner were available because it was out of service temporarily; in addition, the GPS unit at the roof NE corner had problems with satellite signals that caused the alteration of amplitude and loss of certain sections of the displacement history, although the maximum amplitudes of displacement were captured in L component and very close to the maximum in T component.

Response spectra for the eight seismic events in the L and T components, for 5 % of critical damping, are shown in Figure 4. The highest spectral ordinates correspond to the most intense events that have been recorded (11-3 and 12-1). Figure 4 clearly shows the distinct intensity of all seismic events. The dominant long periods

estimated from the horizontal components of the ground motion were determined to range between 1.3 and 2.0 s.

Table 3. Peak acceleration, maximum relative displacements and fundamental frequencies of the building

Event	$A_{max,}$ in cm/s^2		$D_{max,}$ in cm		D(GPS)max, in cm		f, in Hz	
	T	L	T	L	T	L	T	L
11-1	6.7	4.8	<1.0	<1.0	NA	NA	0.59	0.67
11-2	15.0	13.3	1.1	<1.0	NA	NA	0.58	0.64
11-3	78.3	57.7	5.7	3.6	5.8	4.3	0.57	0.62
11-4	2.0	2.0	<1.0	<1.0	<1.0	<1.0	0.58	0.66
12-1	163.0	176.3	14.6	16.5	14.6	15.7	0.55	0.56
12-2	35.3	27.2	3.3	2.5	3.0	2.9	0.55	0.56
12-3	27.9	32.5	2.7	3.1	2.9	3.1	0.55	0.57
13-1	59.4	42.7	5.7	3.8	6.4	3.3	0.55	0.57

NA-Not available data

Spectral analysis of ambient vibration records before and after the rehabilitation showed an increase in the fundamental frequencies of vibration of the structure after the restoration. The increase for the components L, T and torsion were 71, 26 and 48 percent, respectively. On the other hand, the spectral analysis of the records from the first event (11-1, low seismic intensity) showed practically the same fundamentals frequencies as those obtained from an ambient vibration test performed after the rehabilitation completed, with differences of less than one percent.

A spectral analysis was applied to the seismic records listed in Table 1. Figure 5 shows the estimated Fourier spectral ratios between the roof and field sites motions obtained to illustrate the variation of the fundamental frequencies of the L and T components. The shapes of the curves and the intervals of maximum amplitudes vary from one event to another. Furthermore, for some events more than one peak appeared. This suggests that the system exhibited nonlinear behavior during these seismic events. This is clearly noticeable in event 12-1. As a consequence, during event 12-1 the building suffered a reduction of 16 and 6 % of fundamental frequency in L and T components with respect to the first event 11-1 recorded after its rehabilitation. The reduction of stiffness in the building was estimated as well. The values of stiffness loss were 30 and 12%, while the interstory drifts averaged 0.20 and 0.14% for the L and T directions, respectively. It has been found that dynamic properties of the building are sensitive to the intensity of the ground motion. The variation of structural parameters of the system can be attributed mainly to different non-linearity sources of the structure. The identified fundamental frequencies in L and T components are included in Table 3. This frequencies or vibration periods of the building fall within the interval where the dominant periods of the site is located (Figure 4).

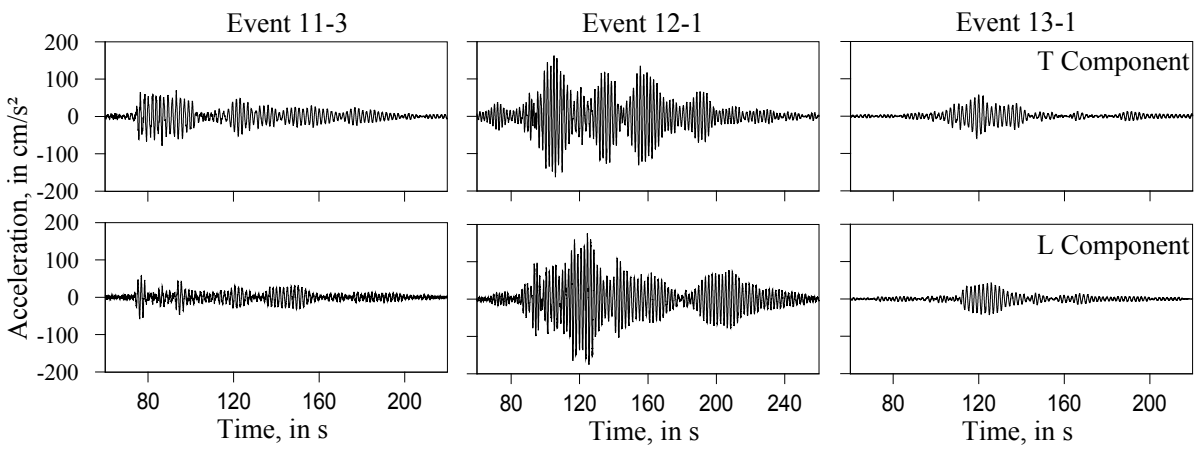
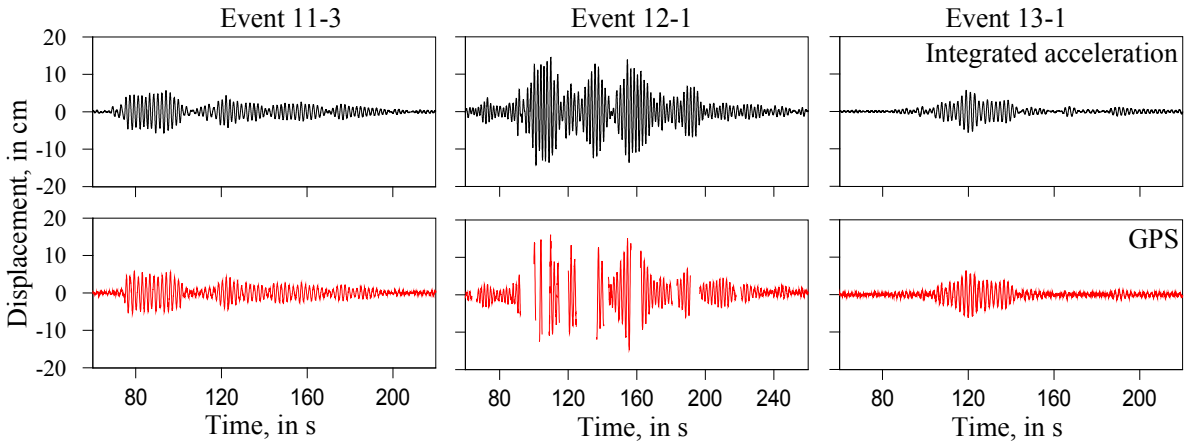
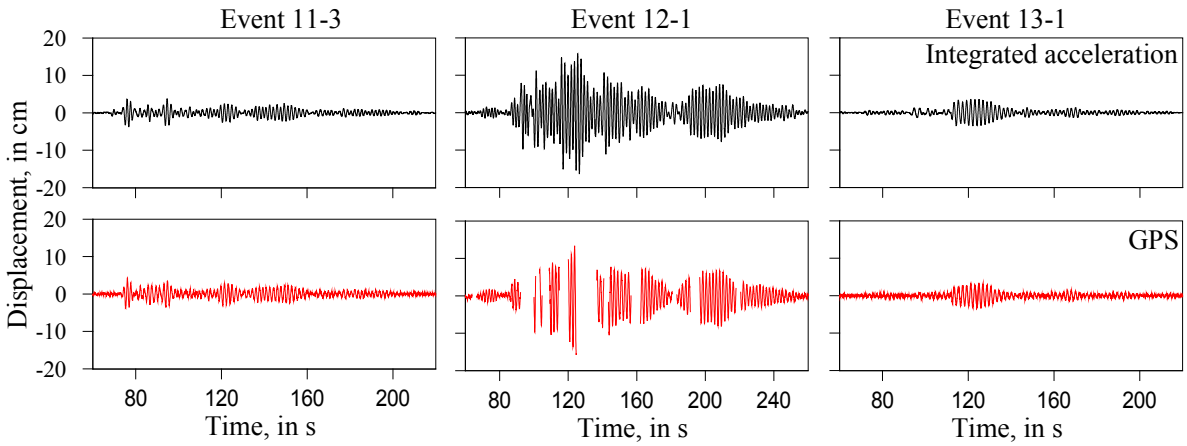


Figure 2. Time-histories of the NE corner roof acceleration of the building



(a) T component



(b) L component

Figure 3. Comparison of time-histories of the NE corner roof displacement's measured with GPS and double-integrated acceleration

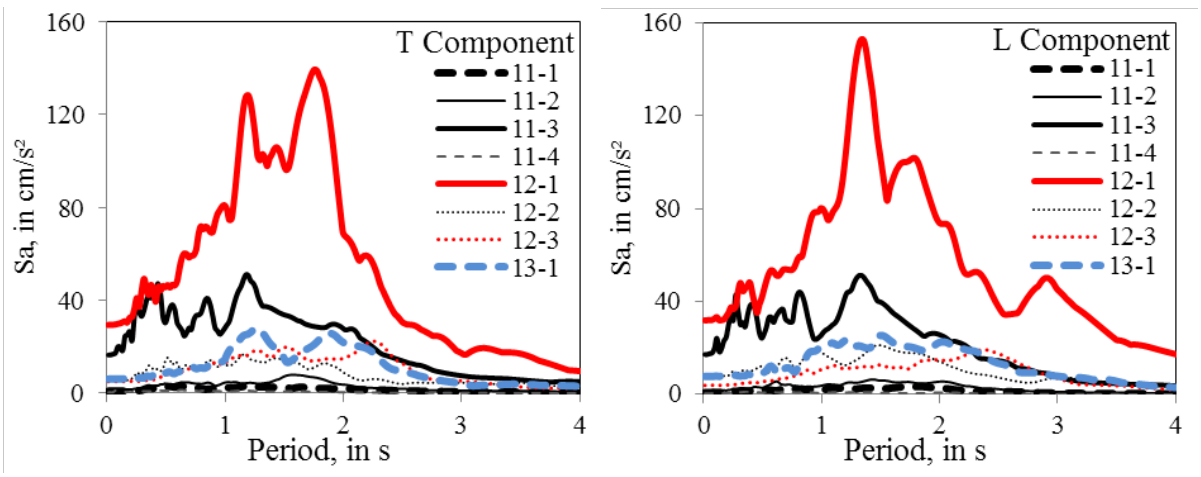
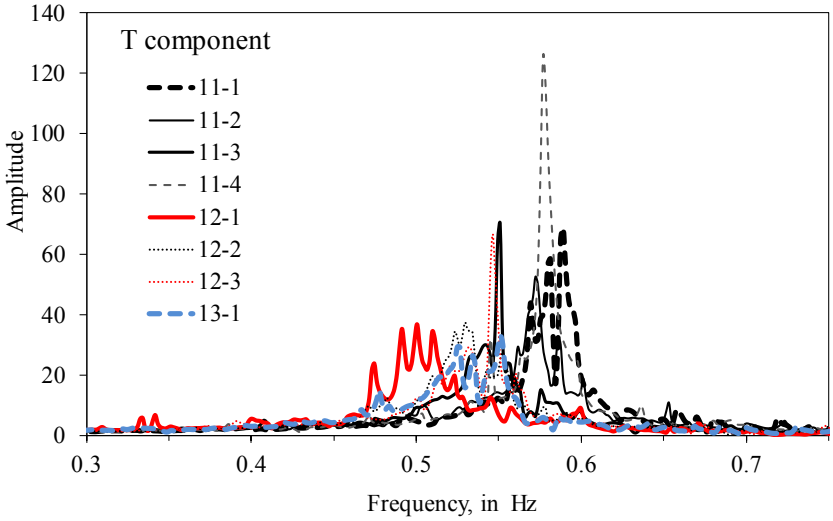


Figure 4. Comparison of response spectra were calculated with the records of the field-surface site



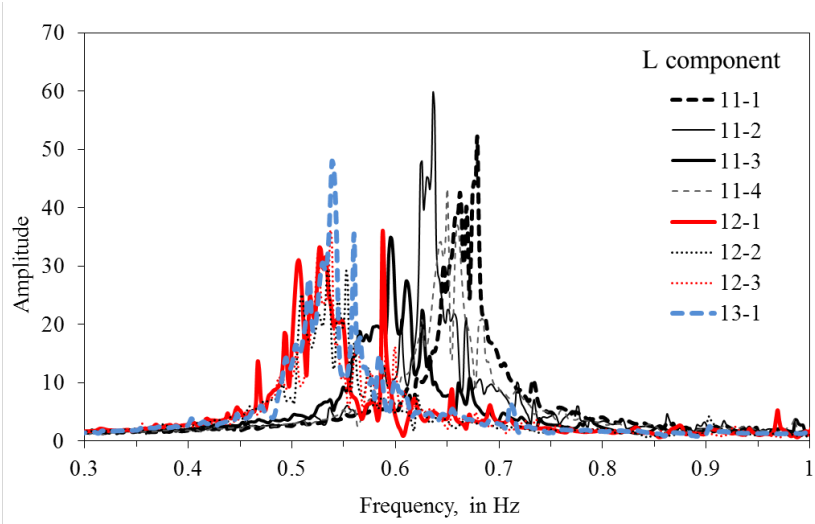


Figure 5. Estimated Fourier spectral ratios between the motion at roof and at field site

PERFORMANCE OF STRUCTURAL WARNING SYSTEM

The assessment of the structural warning system in the building for eight small and moderate earthquakes recorded between 2011 and 2013 is discussed in this section. To estimate the structural condition, the threshold values indicated in Table 2 were used, and the results are shown with colors in Table 4. The SD and VF are calculated from the data of each event in Table 3 and the values are shown in Table 4. It is seen that event 12-1 generally exhibits the highest value for the damage indicators. In particular, the high values of SD (greater than 0.22 %) correspond to the formation of diagonal cracks in masonry walls.

To estimate the structural condition of the building, the warning system compares the values of the five indicators calculated for each seismic event with the thresholds from Table 2. In Table 4 the indicator values together with their corresponding damage level colors are shown. In event 11-3 the warning system detects Yellow because the VF and PGA indicators exceed the level 1 damage thresholds. This damage level suggests possible cracks in the masonry walls. Subsequent inspection showed evidence of that, but the cracks width were small thus it was considered as light damage (Green). In event 11-4 the VF indicator practically regains its initial value, confirming the light damage level of the previous event, and the fact that the initial value was not completely recovered is evidence of deterioration due consecutive seismic events despite its small intensity (Murià-Vila *et al.* 2001). Event 12-1 was classified as Yellow, and almost all the indicators were level 2. The inspection confirmed that the damage level was intermediate. Since event 12-2, the VF indicator has not recovered its initial reference value. Therefore, the damage level remains intermediate for events 12-2, 12-3 and 13-1. In these three events the warning system assigns Green level because they were of small intensity. The results appear in Table 4 and the level assigned for each indicator shows an adequate correlation with the estimated structural damage, except event 11-3.

Table 4. Features of seismic events recorded and status of structural condition in the CCUT building

Event *	PGA, in cm/s^2	I_{Arias}, in cm/s	Cs_{max}, in %		SD_{max}, in %		VF_{max}, in %		Status of level damage
			T	L	T	L	T	L	
11-1 (G)	0.9	0.02	<0.01	<0.01	<0.01	<0.01	0	0	Ligth
11-2 (G)	1.6	0.08	<0.01	<0.01	0.01	<0.01	2	4	Ligth
11-3 (Y)	17.2	2.85	0.01	0.01	0.05	0.04	4	7	Intermediate
11-4 (G)	0.4	<0.01	<0.01	<0.01	<0.01	<0.01	1	2	Ligth
12-1 (Y)	31.8	15.04	0.03	0.05	0.14	0.20	6	16	Intermediate
12-2 (G)	7.9	0.41	<0.01	<0.01	0.03	0.03	6	16	Intermediate
12-3 (G)	5.0	0.42	<0.01	<0.01	0.03	0.04	7	15	Intermediate
13-1 (G)	7.8	0.57	0.01	0.01	0.06	0.05	6	16	Intermediate

* Level of event: (G) Green, (Y) Yellow, (O) Orange

FINAL COMMENTS

In recent years, the tower tilting shows very small variances and the measures after rehabilitation suggest that it has practically stabilized. For structural safety, is important to continue monitory the verticality. The GPS system has been efficient and complements the data obtained with the electronic surveying station. It is expected that the underpinning and the reinforcement of the structure will stabilize further the tilting of the tower.

The displacements measured with the GPS system recorded during small and moderate events show a good performance, except for the event 12-1. This confirms the suggestions made by Çelebi (2013) regarding the potential use of the GPS system in health-monitoring applications in buildings. It is noted that problems with satellite signals similar to those found during event 12-1, which caused the alteration of amplitude and loss of certain sections of the displacement history, cannot be ruled out in future events and warning systems should take into account the implications of this eventual situation.

Spectral analyses of seismic records indicate that the dynamic properties of the system are sensitive to the intensity of the ground motion. The building has suffered damage and the decrease of fundamental frequency suggested stiffness degradation up to 30% in event 12-1. Variation of structural parameters has been mainly attributed to different non-linearity sources in the structure.

An automatic structural warning system to help in the operation and maintenance of the building, which advises on its structural condition based on the recorded event, was presented. The warning system allows the determination, in a few minutes, of the structural condition of the building after the occurrence of an earthquake The system generates quantitative elements pertaining to the dynamic response that facilitate the decision-making process concerning potential repair or evacuation of the building.

The results show that the implementation of the indicators and their correlation with the observed state of damage were appropriate and are supported with results from instrumented buildings obtained in a lapse of up to 20 years. Additional adjustment is not discarded in some of them during the instrumental monitoring process. Reference values can be upgraded depending on the analyzed information of the recorded events and the behavior observed from visual inspection of the building.

The implementation of several indicators in a response evaluation criterion represents an advantage to define the possible structural condition of the building. The indicators complement each other, due to the different level of correlation that each indicator may have on different structural systems. This allows the narrowing of uncertainties when estimating the structural condition of a building.

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