OMA of a School Building Damaged by the Illapel Earthquake (Chile 2015)

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ABSTRACT

The Illapel earthquake struck Chile in 2015, causing fatalities, wounded, missing, and many people left homeless. It also damaged buildings, particularly school buildings. This research refers to the "San Rafael de Rozas" school, located in Illapel (Chile). It was visited after the event, and a simplified OMA (Operational Modal Analysis) of the damaged building was performed; it is required to estimate the modal parameters (mainly, natural periods) of the damaged building. This three-story building was built in 2003 and has an L-plan configuration. The structure consists of RC columns, beams, and slabs; in general, the construction quality was bad. The non-structural components significantly influenced the seismic behavior, chiefly the masonry infill walls. Noticeably, most of them are incomplete (i.e., leave a free space in the top, mainly for lighting); this leads to short columns, which are harmful to the building's seismic capacity. Accordingly, the damage was mainly concentrated in the short columns. For the abovementioned OMA, triaxial acceleration and velocity-sensitive seismographs were installed on the floors of the 3 stories; each measurement lasted 6 minutes at a sampling frequency of 512 Hz.

Keywords: OMA of Damaged Buildings, Chile, Illapel Earthquake, School Buildings, Observed Damage, Short Columns

1. INTRODUCTION

The $M_w = 8.4$ Illapel earthquake struck central Chile in September 2015, causing 15 deaths, 6 missing, and thousands of affected people. Moreover, it severely damaged many facilities and buildings, particularly school buildings [1,2]. This research refers to the most damaged school building, namely "San Rafael de Rozas", located in Illapel (Coquimbo region, central Chile). It was deeply inspected

after the event, including soil geophysical measurements and simplified OMA (Operational Modal Analysis) of the damaged building. The final objective of the research is to reproduce by numerical simulation the observed damage; the research approach consists of: (i) estimating the exciting seismic accelerograms (two horizontal components) from the seismic records in close seismological stations, (ii) developing a nonlinear numerical model of the building seismic behavior (including the influence of the site soil), and (iii) using it to perform seismic dynamic analyses under the excitation of the estimated accelerograms. The results of these analyses are expected to match the observed damage; this will corroborate this important capability of numerical modeling. The OMA performed is required to estimate the modal parameters (mainly, natural periods) of the damaged building; this is the reason for this study.

This three-story building was built in 2003 and has an L-plan configuration. Three wings make up the building, being separated by seismic joints as their seismic behaviors are rather independent; this study focuses on the central wing (including the corner of the L) as it was the most damaged during the earthquake. The structure consists of RC columns, beams, and slabs; in general, the construction quality is rather poor. The non-structural components played a highly relevant role in the lateral seismic structural behavior, particularly the masonry infill walls. Noticeably, most of them are incomplete (i.e., do not encompass the full height of the columns, mainly for lighting purposes); this leads to the existence of many short (captive) columns, being extremely detrimental to the building's seismic performance [3]. In fact, the damage was mainly concentrated in some of the short columns of the first and second levels; this significantly altered the modal characteristics of the building, most precisely, the natural periods were significantly elongated due to the lateral (horizontal) stiffness reduction.

For the abovementioned OMA, triaxial acceleration and velocity-sensitive seismographs were installed in the centers of gravity of the floors of the 3 stories; each measurement lasted 6 minutes at a sampling frequency of 512 Hz. Because of the restrictions related to the accessibility of some areas of the building, it was impossible to locate additional instrumentation in other areas of each floor. The natural frequencies of the building in NS and EW directions were estimated.

2. "SAN RAFAEL DE ROZAS" SCHOOL BUILDING

This school building is located in Illapel and was built in 2003; Figure 1 describes the overall characteristics of the three-story L-shaped building. This construction was inspected on October 8, 2015.
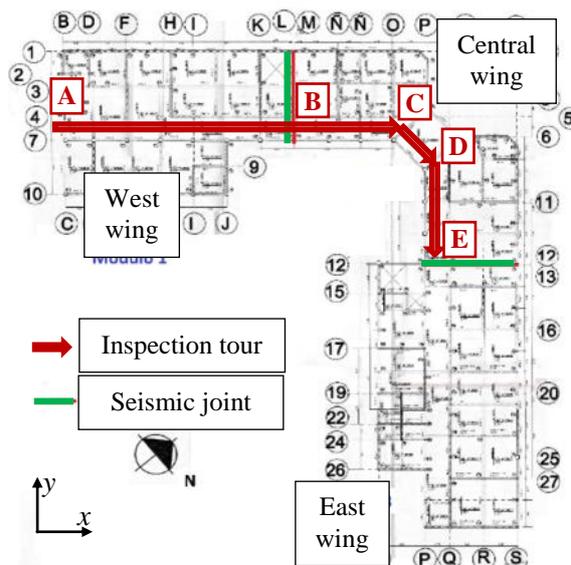
(a) General view   (b) Plan Layout

Figure 1. "San Rafael de Rozas" school building (Illapel)

The plan layout displayed in Figure 1.b shows that three wings make up the building; they are separated by seismic joints (indicated with green lines). The central wing has three stories and an L-shaped plan, and the external wings have two stories and are rectangular, being attached to both ends of the central one. Figure 1.a displays the main entrance to the building, which is located in the corner of the central wing. This paper focuses mainly on the central wing, which was the most damaged during the earthquake. The total height of the buildings is approximately 9.5 m and the plan area of the central wing is approximately 360 m^2. The structure consists of 3-D RC moment-resisting frames consisting of columns, beams, and slabs. Infill walls are made of unconfined and unreinforced masonry [4]. Noticeably, most of them are incomplete (i.e., do not encompass the full height of the story leaving a top opening useful for lighting); this leads to many short (captive) columns, extremely detrimental to the building's seismic performance. In fact, the damage was mainly concentrated in some of the short columns of the first and second levels. The construction quality is generally poor, and most probably does not fulfill the Chilean seismic design code [5].

Figure 2 displays a more detailed plan view of the structure and the infill walls of the central wing 2nd story; the complete/incomplete walls (reaching/not reaching the story height) are represented in dark/light blue.
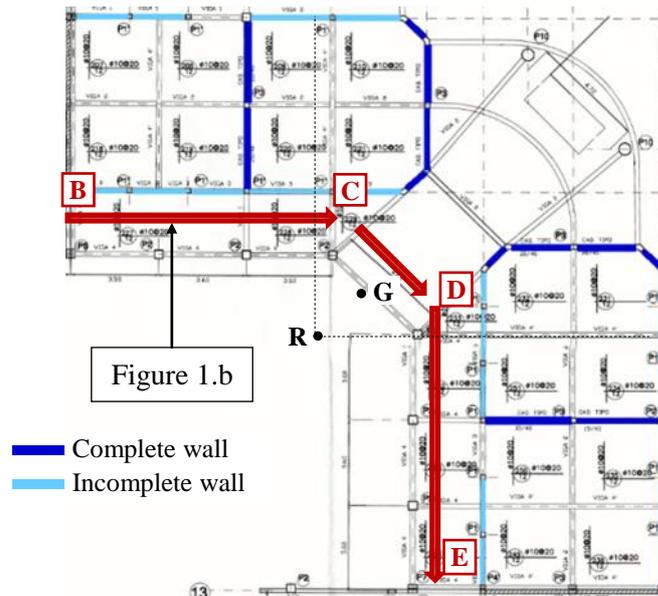
Figure 2. Structural plan layout of the second story of the central wing (Illapel)

Figure 2 shows the approximate positions of points R (center of rotation, or rigidity) and G (center of gravity) of the second story; they are determined using the numerical model described in section 5 (for the undamaged building but with the cracked members). As for R, its position is selected mainly based on the complete walls (dark blue); as they are oriented in orthogonal directions, that point is located at the intersection between the center lines parallel to walls in NW and NE directions. As there is an important eccentricity between points R and G, the central wing has a great asymmetry in plan. Therefore, a relevant torsion motion is to be expected; this can be understood as a rather poor seismic design. Finally, the red arrows describe the itinerary (B-E, Figure 1.b) of the inspection tour, including the most severe damage observed.

3.   FIELD INSPECTION OF THE DAMAGED BUILDING

This section describes the damage observed during the post-event visit. Figure 2 illustrates the itinerary (B-E) inside the central wing and Figure 3 displays images of the observed damage in the segment BC of this circuit. Figure 3.a exhibits an overview of segment BC and the following Figures (Figure 3.b through Figure 3.g) present details of the observed damage. The cases depicted in Figure 3 are representative examples of many similar situations occurring in other locations inside the building. Figure 3.a shows that the incomplete interior wall has five short columns connecting this wall to the upper slab; also, the two-door jambs play a similar role. Figure 3.b to Figure 3.f confirm that all columns experienced similar levels of damage; the last (fifth) column did not exhibit any apparent damage, but Figure 3.g shows that the sill wall separated from the adjoining column, thus breaking its captive condition. This trend is expected in any short column alignment, as they all experienced roughly the same transverse distortion; however, the drift appeared in the direction of the wall and also in the orthogonal.
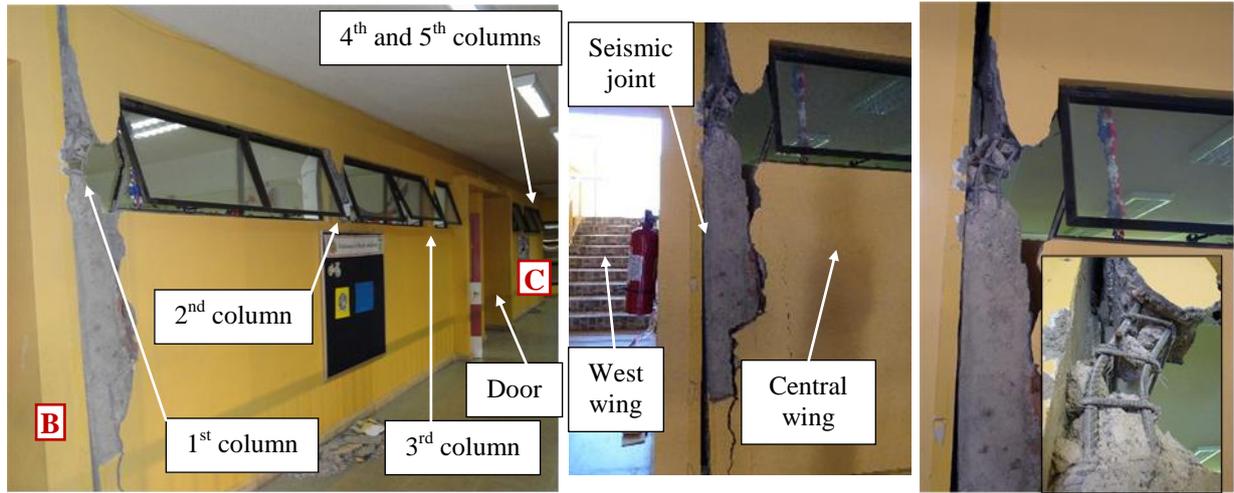
(a) General view

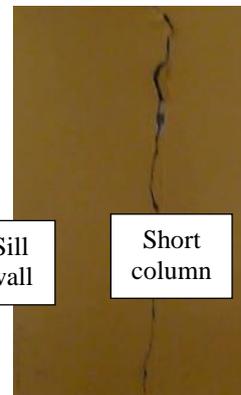(b) First column (general view)

(c) First column (details)

(d) Second column

(e) Third column

(f) Fourth column

(g) Fifth column

Figure 3. Observed damage to segment BC (Figure 2) of the inspection itinerary. Central wing

Figure 3.b to Figure 3.f indicate that all the columns are rather similar; externally their section is 20 cm × 30 cm, and their "core" (concrete kern confined by the transverse reinforcement) is approximately 12 cm × 12 cm. The longitudinal reinforcement consists of four 16 mm diameter bars and 6- or 8-mm stirrups separated 12 cm; this core is insufficient, being smaller than indicated in the design (square of 17 cm on each side), and its shear and bending strength is severely diminished. On the other hand, the quality of the concrete seemed pretty bad; some important cavities were even observed. Finally, it should be highlighted that the observed damage revealed that relevant torsion occurred; this consideration is because the farthest column from the center of rotation (Figure 2) exhibited the worst damage and Figure 3 shows significant horizontal displacements orthogonal to the wall plane. This trend corroborates the considerations from Figure 2. Figure 3.b to Figure 3.f show that the columns failed by shear rather than bending; this is expected, as they are highly short (stocky). The conditions of each column are discussed in more depth next. Figure 3.b and Figure 3.c refer to the first column, apparently the most damaged. Figure 3.b shows that this column is adjacent to the seismic joint that separates the west and central wings. Figure 3.b and Figure 3.c highlight that the lowest segment of the column rotated to the left until it collided with the west wing and generated some pounding; the upper column segment (the most damaged) failed due to spalling and subsequent destruction of the concrete core and buckling of the longitudinal rebars. Noticeably, in the first column, the transverse reinforcement was made of 8 mm diameter bars and did not fail. On the other hand, the first column exhibited lateral displacement in both horizontal directions; this is coherent with the high eccentricity

between the centers of mass and of rotation displayed in Figure 2. Regarding the other columns, Figure 3.d shows that the second column had a shear failure; the separation of the stirrups in the affected segment was particularly high. Analogously, Figure 3.e and Figure 3.f confirm that the third and fourth columns also failed due to shear, although without breaking the transverse reinforcement.

4. OPERATIONAL MODAL ANALYSIS OF THE SCHOOL BUILDING

A simplified Operational Modal Analysis (OMA) was performed on the damaged school building [6]. Triaxial acceleration and velocity-sensitive seismographs were installed in the centers of gravity of the floors of the 1st, 2nd, and 3rd stories (belonging roughly to the same vertical line); each measurement lasted 6 minutes and the sampling frequency was 512 Hz. Figure 4 displays the plots of the H_i/H_1 spectral horizontal components vs. the response frequency (logarithmic scale); H_i corresponds to the spectral acceleration component of the i-th story floor, and H_1 refers to the 1st story floor (ground). Figure 4 presents the ratios H_i/H_1 for $i = 1$ (green), $i = 2$ (red), and $i = 3$ (blue); obviously, for $i = 1$ the plot is merely a horizontal line ($H_1/H_1 = 1$).
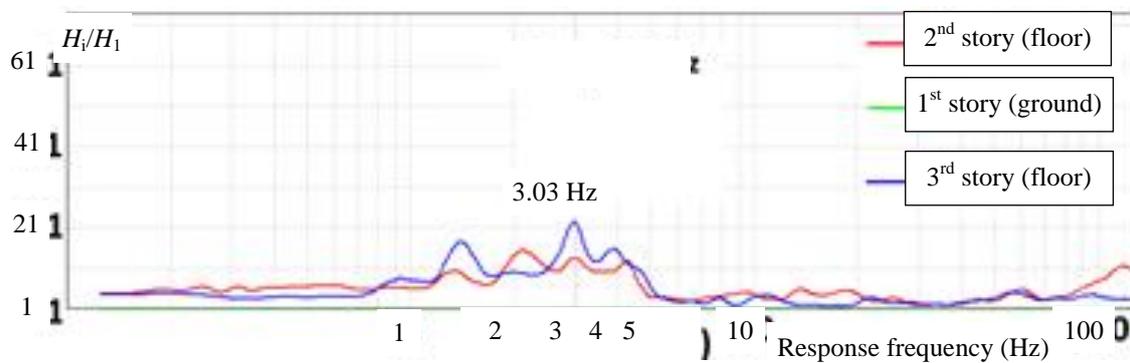
(a) NS direction (Figure 1.b)
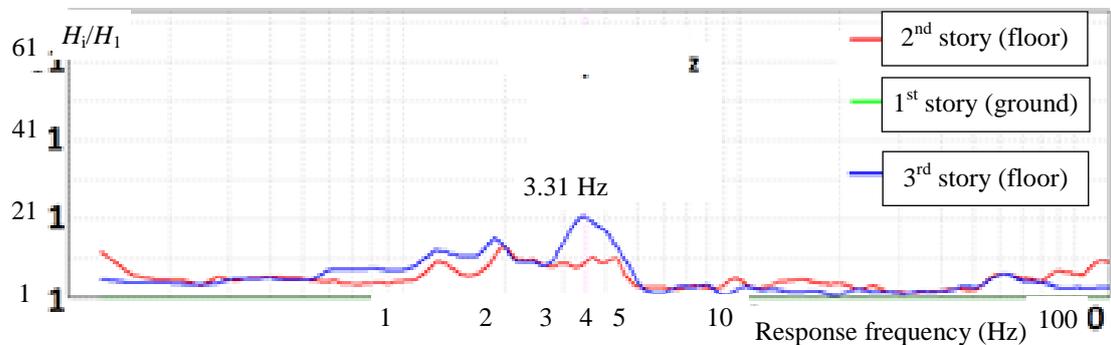
(b) EW direction (Figure 1.b)

Figure 4. Measured (OMA) vibration amplitude ("San Rafael de Rozas" school building)

The important amplifications in Figure 4 indicate that the building is highly flexible in the horizontal directions; the estimated building's natural frequencies corroborate this. This estimation (from the plots in Figure 4) is far from obvious due to the uncertainties involved in the measurements and, mainly, to the difficulty in distinguishing the peaks corresponding to the natural frequencies of the building (for global vibration modes) from those referring to either local modes or input frequencies. Regarding this latter issue, it is assumed that peaks that appear simultaneously (i.e., for the same frequency) in the 2nd and 3rd stories correspond most probably to a global mode in the considered lateral direction.

- **NS direction**. The plots in Figure 4.a exhibit the highest peak (in both stories, H_2/H_1 and H_3/H_1) for a frequency approximately equal to 3.03 Hz (period 0.330 s); as the spectral amplitude is

higher for the 3rd story, it can be presumed that the shape of this mode is expected for the first one. To further ground this estimation, it should be kept in mind that the peaks for lower frequencies are shorter and do not affect both stories.

- **EW direction**. In the EW direction, the highest peak of the plot in Figure 4.b corresponds to a frequency approximately equal to 3.31 Hz (period 0.302 s).

Comparison between the periods in both directions shows a high similarity; this matches the information in Figure 2.

5. NUMERICAL MODAL ANALYSIS OF THE SCHOOL BUILDING

The seismic behavior of the "San Rafael de Rozas" school building is numerically simulated with a 3-D model of its central wing (Figure 1 and Figure 2) implemented in SAP2000 version 19.2.1 [7]. The main objective of this model is to reproduce the building's observed damage; it includes separate versions for its undamaged and damaged states. Figure 5 displays a 3-D view of the model. As the non-structural components played a highly relevant role in the lateral seismic structural behavior (particularly the masonry infill walls), they are incorporated into the model.
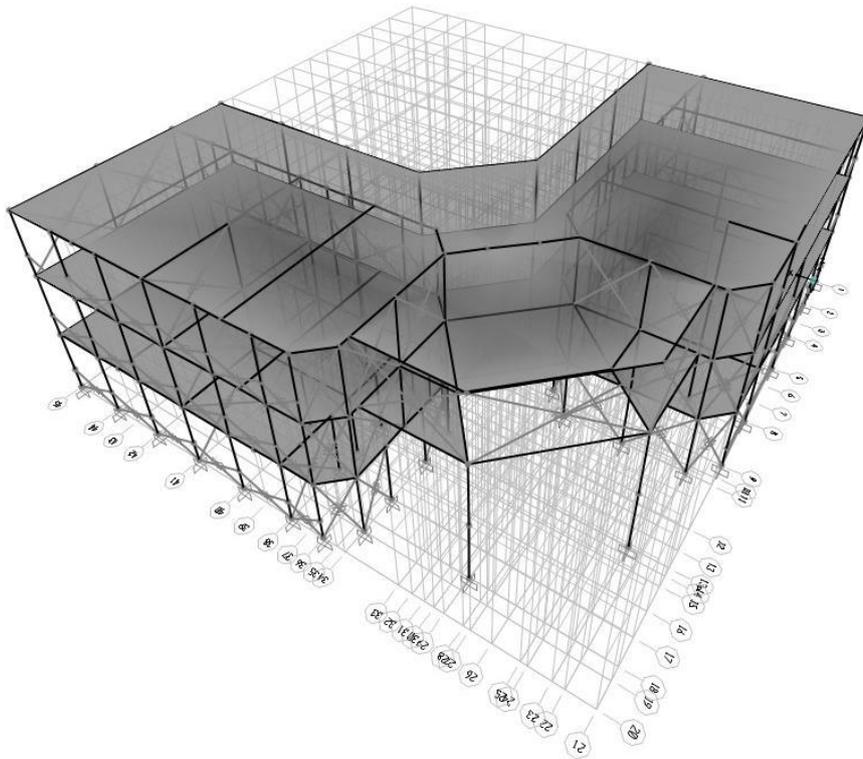

Figure 5. Numerical model of the central wing of the "San Rafael de Rozas" school building

Figure 5 shows that the model includes the main structural and non-structural elements, namely columns (even the short ones), slabs, and masonry infill walls; as only the short columns and the complete walls (those spanning the free height of the story) exhibited relevant damage, their behavior is modeled as nonlinear, while that of the other members is linear.

Table 1 displays the modal parameters (natural periods and modal mass participation ratios) of the central wing of the San Rafael de Rozas school building. Table 1 contains results obtained in two situations: uncracked members (the axial, shear, and bending stiffnesses of the structural elements are determined for their gross cross sections, i.e., not accounting for the reduction due to cracking of the tensioned concrete) and cracked members (the bending stiffnesses are reduced as indicated in [8]). These reduction coefficients are 0.35 I_g (vertical direction) for beams, 0.25 I_g (vertical direction) for

slabs, 0.70 I_g (any horizontal direction) for columns, and 0.70 I_g (in-plane bending) for walls; I_g is the moment of inertia of the gross (integer) sections. Each cell in Table 1 contains two values, the left/right ones refer to the initial/final (undamaged/damaged) status of the structure; in the final situation, the damaged short columns are replaced with pin-ended (hinged) columns (i.e. retaining their vertical capacity but without any lateral stiffness).

Table 1. Calculated modal parameters (*) of the central wing of the San Rafael de Rozas school building

Mode No.	Uncracked members (gross sections)				Cracked members (reduced sections)			
	Period (s)	UX (%)	UY (%)	RZ (%)	Period (s)	UX (%)	UY (%)	RZ (%)
1	0.39/0.47	0.07/0.01	0.04/0.01	**0.78/0.85**	0.42/0.48	0.03/0.01	0.01/0.04	**0.84/0.83**
2	0.33/0.37	0.25/**0.75**	**0.64**/0.10	0.00/0.01	0.36/0.40	0.04/**0.55**	**0.86**/0.31	0.00/0.00
3	0.32/0.36	**0.56**/0.09	0.22/**0.78**	0.11/0.02	0.34/0.39	**0.80**/0.29	0.03/**0.56**	0.04/0.01

* Left/right values correspond to undamaged/damaged building, respectively

In Table 1, UX and UY refer to the modal mass percentages in the X and Y directions, respectively (Figure 1.b); RZ represents the modal rotational mass (moment of inertia) percentage with respect to the vertical axis (torsion). Table 1 shows a regular and expected behavior; for both cracked and uncracked results the first mode is predominantly rotational, while modes 2 and 3 are mainly translational corresponding to Y and X directions, respectively. The fact that the first mode is mainly rotational is commonly recognized as a rather poor seismic design because torsion is highly damaging, particularly for the supporting elements (walls and columns) located far from the rotation center. Comparison between the cracked and uncracked results shows that concrete cracking generates a relevant period elongation (7.71, 9.45, and 8.54% for the first three modes, respectively); regarding the mass percentages, the cracked modes exhibit a more regular behavior, with smaller "crossed" values. This trend can be understood as a smaller influence of the eccentricity between the mass (G) and rotation (R) centers (Figure 2). Comparison between the initial and final results shows that the seismic damage significantly altered the modal characteristics of the building, most precisely, the natural periods were significantly elongated due to the lateral (horizontal) stiffness reduction; also, the X and Y modes altered their order (this is expected, given the high proximity between their periods).

6. COMPARISON BETWEEN EXPERIMENTAL (OMA) AND NUMERICAL MODAL PARAMETERS OF THE DAMAGED SCHOOL BUILDING

This section compares the natural frequencies of the damaged building obtained from OMA (section 4) and numerical simulation (section 5); more precisely, this comparison refers to the frequencies of the 1st mode in the X and Y directions. The OMA described in section 4 provides the frequencies in the **NS** and **EW** directions (Figure 1.b), being **3.03** and **3.31** Hz, respectively; given the eccentricity between the mass and rotation centers (Figure 2), these frequencies shall be influenced by the first three modes (Table 1). Regarding the numerical results, as the excitation is so weak that the cracks do not open, the uncracked sectional stiffnesses (Table 1) should be considered; in brief, the numerical frequencies of the 1st mode in the **X** and **Y** directions are **2.70** and **2.78** Hz (0.37 and 0.36 s), respectively.

Comparison between measured (OMA) natural frequencies (3.03 and 3.31 Hz in NS and EW directions, respectively, see Figure 4) and the numerical ones (2.70 and 2.78 Hz in X and Y directions, respectively, see Table 1) shows a reasonable agreement; more precisely, the OMA results correspond to a slightly higher stiffness. The observed differences can be explained by inaccuracies in the OMA measurements and, mainly, by simplifications and uncertainties in the numerical analyses; in this sense, the higher stiffness in the experimental results might be due to a possible stiffening due to the continuity of the slabs through the seismic joints (thus provided a higher torsion strength). Noticeably, some production blueprints contained contradictory information regarding the seismic joints in the slabs.

7. CONCLUSIONS

This paper refers to the "San Rafael de Rozas" school building located in Illapel, Chile; it was damaged by the Illapel earthquake, $M_w = 8.4$, 2015. This research compares the lateral natural frequencies measured from a simplified OMA and simulated with a numerical model of the structural behavior. The agreement between the experimental (OMA) and numerical values is rather satisfactory, with a difference of around 16%; this corroborates the reliability of the study performed. This difference can be due to inaccuracies in the measurements and, mainly, by simplifications and uncertainties in the numerical analyses.

This paper is part of a wider research effort aiming to simulate numerically the building's observed damage.

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