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RESEARCH PAPER

Assessment of Site Response and Building Damage Following the Mw 6.4 Copiapó Earthquake

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ABSTRACT

On June 6, 2025, an inslab earthquake of Mw 6.4 occurred in the north-central region of Chile, near the city of Copiapó, within the Atacama seismic gap. Despite its moderate magnitude, the event produced strong ground motions with recorded accelerations exceeding 0.4 g and widespread nonstructural damage. Due to these unusual consequences, we conducted a multidisciplinary field study, collecting in situ data, which provides an opportunity to characterize both local site conditions and structural behavior in a mature seismic gap region. Shallow shear-wave velocity profiles and predominant site periods were derived from ambient-vibration measurements to evaluate site amplification effects in Copiapó. Damage surveys revealed widespread nonstructural damage in designated evacuation zones, significantly impacting the functionality of numerous facilities. Finally, using ambient-vibration measurements, we estimated the structural properties of nine buildings (five damaged and four undamaged) of a residential housing complex located within the affected area. Despite their similar architecture and design, we identified systematic differences in the dynamic properties of the damaged buildings compared to the undamaged ones. On average, damaged buildings presented approximately 20% larger fundamental vibration periods and 10% higher damping ratios than undamaged buildings. The findings of this field survey provide insight into the vulnerability of buildings under moderate seismic events, whose seismic demand remains below that expected during a major earthquake.

1 | Introduction

Chile sits on the tectonic boundary where the Nazca Plate subducts under the South American Plate at a rate of approximately 6–7 cm/year (Jarrin et al. 2022), making it one of the world's most seismically active countries. This subduction causes frequent and large interface and inslab earthquakes that control the seismic hazard across much of the country. Examples of interface megathrust events in Chile include the Mw 9.5 Valdivia earthquake in 1960 (e.g., Kanamori and Cipar 1974; Cifuentes 1989) and the Mw 8.8 Maule earthquake in 2010 (e.g., Vigny et al. 2011). However, Chile has also suffered destructive inslab earthquakes, such as the Mw 7.8 Tarapacá 2005 (e.g., Delouis and Legrand 2007) and the

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Mw 7.8 Chillán 1939 (Beck et al. 1998) events. Particularly in north-central Chile, there is a well-studied seismic gap (Métois et al. 2013; Yáñez-Cuadra et al. 2022; González-Vidal et al. 2023) since the last large-magnitude event in 1922 (Mw 8.5–8.6) (Kanamori et al. 2019; Vigny et al. 2024). This anomaly has drawn significant interest from national and international research groups studying the seismic behavior at these latitudes and is of particular importance considering how prepared citizens, authorities, and decision-makers are for a potential major earthquake in the Atacama region.

On June 6, 2025, at 13:15 local time (17:15 UTC), a moderate inslab Mw 6.4 earthquake struck the Atacama region, mainly affecting its capital city, Copiapó. According to the Global Centroid Moment Tensor (GCMT) project (Ekström et al. 2012), the epicenter was located at 26.62°S 70.57°W, with a focal depth of 80.8 km. Despite its moderate magnitude, the event produced strong shaking in the region. For example, a peak ground acceleration (PGA) of 0.44 g was recorded at a station in Copiapó, located 116.4 km away from the hypocenter. As a result, several buildings suffered various levels of structural and nonstructural damage.

This study presents the results of a field survey conducted following the Mw 6.4 Copiapó earthquake. The investigation includes an assessment of local site conditions and a detailed characterization of building damage throughout the affected area. Ambient-vibration measurements were performed at six sites within Copiapó and one site in Chañaral to estimate shallow shear-wave velocity profiles and predominant site periods. Additionally, nine buildings within the housing complex were surveyed using ambient-vibration measurements, including free-field recordings to characterize site (geotechnical) conditions (determination of V_{S30}) and recordings within the buildings to identify their dynamic properties. Although these buildings had similar design, construction, and architectural features, they exhibited notably different levels of performance and damage. Finally, despite its moderate magnitude and considerable depth, the 2025 Mw 6.4 Copiapó inslab earthquake and its ground-motion characteristics provide valuable insights into the potential impacts on populated areas within a region prone to significantly higher seismic demands.

2 | Seismological Setting

North-central Chile lies within the active Nazca–South America subduction zone, where a diversity of seismic sources occur daily, including widespread intermediate-depth inslab earthquakes with depth ranging from 70 to 120 km (hereafter inslab events). Several destructive inslab events have occurred throughout the 20th and 21st centuries, including the 1939 Mw 7.8 Chillán (the deadliest earthquake in Chile’s history), 1950 Mw 8.0 Calama, 1965 Mw 7.4 La Ligua, 1997 Mw 7.1 Punitaqui, and 2005 Mw 7.8 Tarapacá earthquakes (e.g., Beck et al. 1998; Ruiz and Madariaga 2018). However, other recent M6-class earthquakes, such as the 2019 Mw 6.7 Coquimbo earthquake (Ruiz et al. 2019), also caused high PGAs and macroseismic intensities reported in nearby cities. These examples demonstrate the importance of studying inslab events, despite their magnitude, which can lead to significant seismic demand in populated cities or critical infrastructure.

The Atacama segment in north-central Chile has hosted major historical megathrust earthquakes (Figure 1); however, no event with a magnitude greater than M8.0 has occurred since 1922 (Comte et al. 2002; Vigny et al. 2024). Therefore, this region is known as a mature seismic gap (Lay and Nishenko 2022). The continuous monitoring of seismic and geodetic records has enabled us to study this region more thoroughly and identify a diversity of seismic and aseismic processes with complex interactions along and nearby the interface (e.g., Ojeda et al. 2023; Klein et al. 2023; Münchmeyer et al. 2025). Furthermore, the occurrence of both large interface and moderate inslab earthquakes, particularly in northern Chile, indicates a complex mechanical interaction between these faulting styles (e.g., Bouchon et al. 2016, 2018; Jara et al. 2017). Hence, this region is crucial for understanding stress transfer, slab deformation, and the potential reactivation of locked or partially locked segments along the Chilean megathrust.

The June 6, 2025, Mw 6.4 Copiapó earthquake was reported by national agencies, such as the Chilean National Seismological Center (Centro Sismológico Nacional [CSN]), and international agencies, including the GCMT project and the National Earthquake Information Center (NEIC) from the US Geological Survey. The hypocentral locations differ among the seismological centers (e.g., CSN: lon 70.14°W, lat 26.87°S, depth 66 km; NEIC: lon 70.404°W, lat 26.639°S, depth 75 km), particularly in depth. However, moment tensor solutions from GCMT and NEIC are consistent in terms of their location (Figure 1) and depth (GCMT: 80.8 km, NEIC: 80.4 km), indicating a normal-faulting mechanism with east–west-oriented nodal planes and a significant nondouble-couple component (i.e., a high compensated linear vector dipole [CLVD]), which is atypical for the prevailing stress regime in this region, with expected approximately north–south faulting within the slab from inherited fault fabrics (e.g., Pardo et al. 2002). Furthermore, given its focal depth within the Nazca Plate, this earthquake occurred below the transitional zone of partial interface coupling (pink areas in Figure 1).

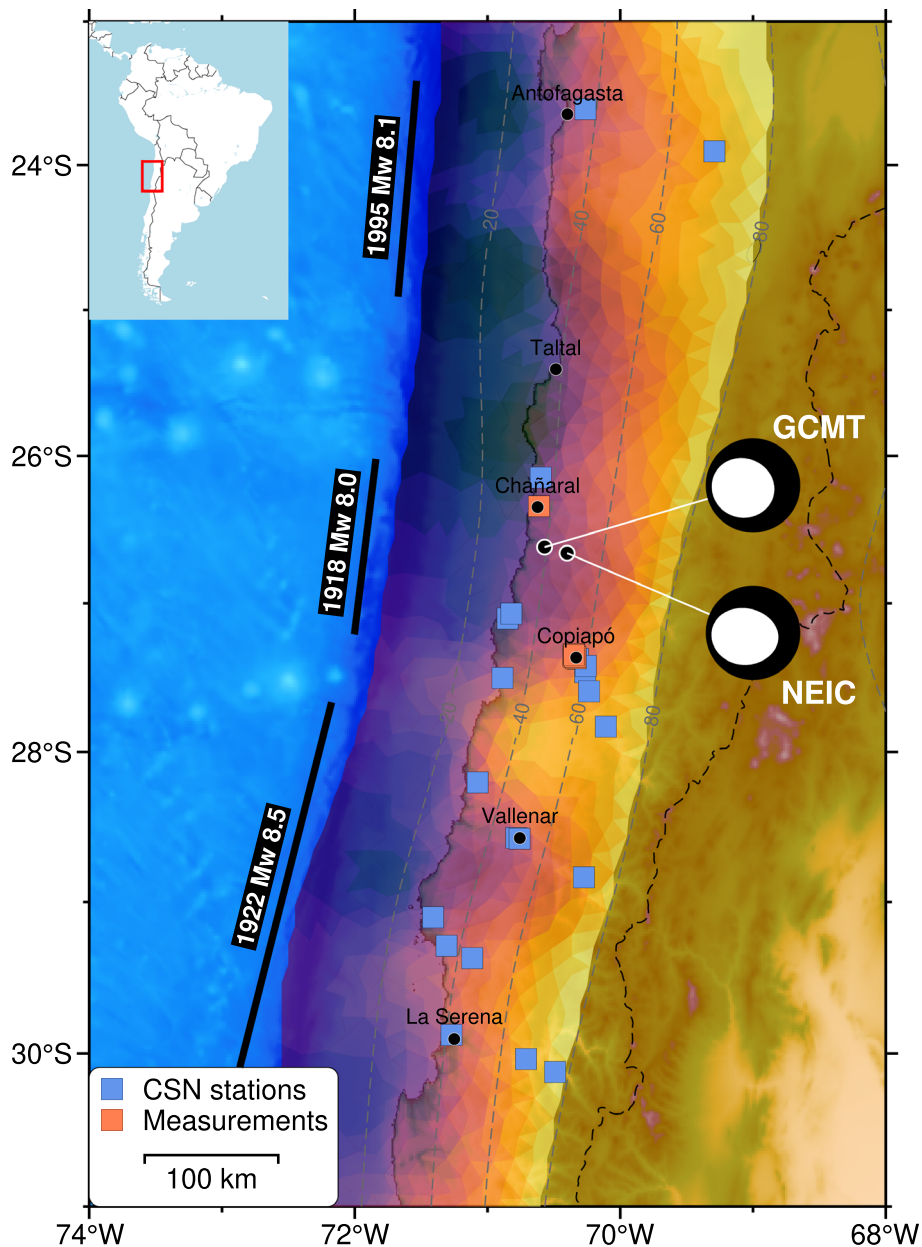


FIGURE 1 | Seismotectonic context of north-central Chile. The 2025 Mw 6.4 inslab event is indicated by its centroid locations (black circle) and moment tensor solutions from the Global Centroid Moment Tensor (GCMT) and the National Earthquake Information Center (NEIC). The Atacama region is recognized as a seismic gap, characterized by locked regions (purple and black areas) with high coupling, in contrast to low coupling zones (yellow areas). The length of large subduction earthquakes Mw $\geq$ 8.0 is indicated for the 1922 Mw 8.5, 1918 Mw 8.0 (Vigny et al. 2024), and 1995 Mw 8.1 (Pritchard and Simons 2006) earthquakes. Blue and orange squares indicate the CSN stations and the sites where geophysical measurements were conducted, respectively. Slab depth isocontour, every 20 km, is shown in gray dashed lines (Hayes et al. 2018). The main cities mentioned in the manuscript are labeled for reference.

3 | Recorded Ground Motions

Twenty-three strong-motion stations recorded the Copiapó earthquake (see Figure S1), as reported by the CSN (Barrientos 2018). The records span hypocentral distances of approximately 90–370 km (Figure 2). Strong-motion records were processed following the workflow of Bastías and Montalva (2016) and Bastías et al. (2026). The highest PGA was 0.44 g at station A06C–HNE (Teletón Copiapó), with a hypocentral distance of 116.4 km, whereas the highest peak ground velocity (PGV) was 14.5 cm/s at station A07C–HNN (23rd Military Regiment Copiapó) with a hypocentral distance of 118.9 km. In contrast, stations located closest to the source (AC01, A30C, and A02C) exhibit substantially lower PGA values (see Figure S2). Figure 2 presents distance-attenuation trends for PGA and PGV. The recorded peak ground values are systematically higher than those predicted by available subduction-zone ground-motion models. The models

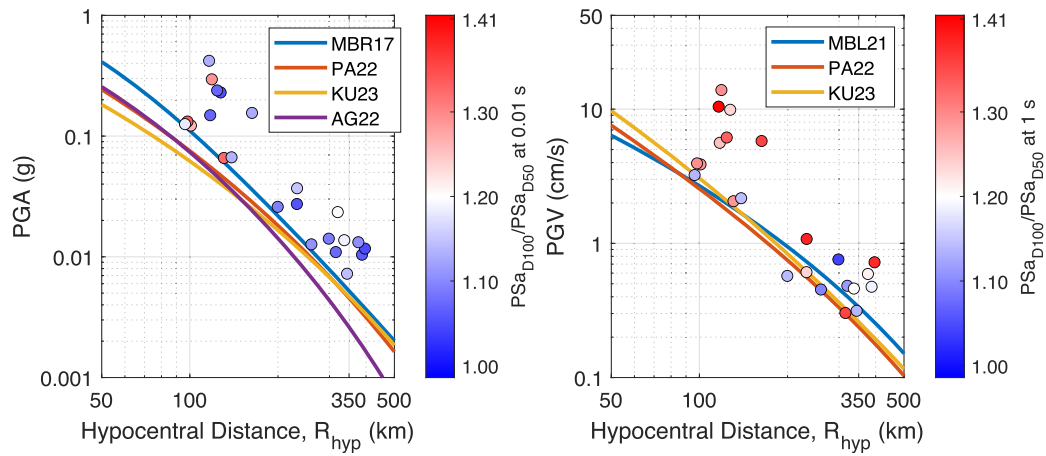


FIGURE 2 | Distance attenuation of recorded PGA and PGV: four inslab-subduction GMM medians (lines) compared with observations (circles). Color bar denotes polarity based on the ratio $PSa_{RotD100}/PSa_{RotD50}$.

used in the analysis were MBR17 (Montalva et al. 2017), PA22 (Parker et al. 2022), KU23 (Kuehn et al. 2023), and AG22 (Abrahamson and Gulerce 2022) for PGA and MBL21 (Montalva et al. 2021), PA22 (Parker et al. 2022), and KU23 (Kuehn et al. 2023) for PGV. To analyze the polarity of the recorded ground motions, we used the ratio $PSa_{RotD100}/PSa_{RotD50}$: An unpolarized record yields a ratio of 1.0, whereas a perfectly polarized record yields a ratio of 1.41 (Boore 2010). Polarized motions (red markers) are more frequent at longer periods (~ 1 s); that is, a preferred spectral orientation along a single azimuth is observed in this case.

Residual analyses were performed for spectral periods ranging from 0.01 to 10 s, considering the site conditions at each station. The corresponding means and standard deviations are shown in Figure 3. In all the cases, residuals are positive, indicating systematic underprediction of the models for this earthquake of approximately $+1\sigma$. The largest residuals occur at periods near 1.0 s. Available GMMs exhibit a systematic underprediction across the entire analyzed range of distances and spectral periods. While such deviations are not uncommon for individual earthquakes, systematic positive residuals across periods and distances suggest that specific features of this event may not be fully captured by current

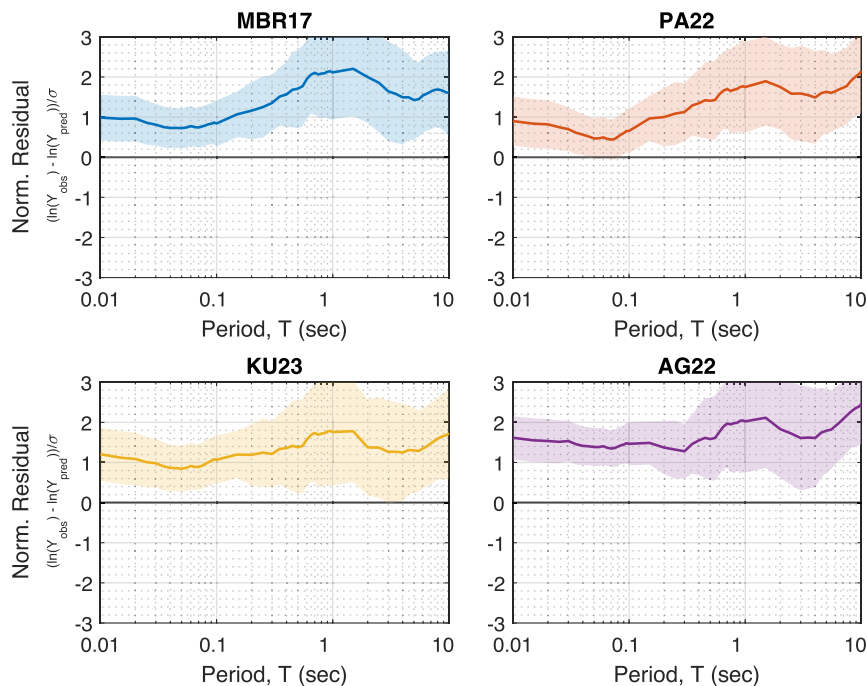


FIGURE 3 | Spectral acceleration residuals for four GMMs over 0.01–10 s. Medians are shown as lines, and shaded areas denote the 16/84th percentiles of the observed residuals.

ground-motion models. However, a detailed analysis of the potential causes of this deviation is beyond the scope of this article.

Finally, we computed the log-likelihood (LLH) score (Scherbaum et al. 2009) and a Euclidean-distance-based ranking (EDR) (Kale and Akkar 2013) to compare the predictive performance of the GMMs (Figure 4). Three of the four ground-motion models (MBR17, PA22, and KU23) exhibit comparable predictive performance across the analyzed spectral periods, with good agreement up to 0.2 s ($LLH < 2$). Beyond this period, the LLH and EDR metrics indicate degraded predictability, driven by a significant misfit in the predicted means and a large standard deviation of the residuals (McNamara et al. 2019). The AG22 model shows limited predictive capacity ($LLH > 2$) across all periods for the observed data.

4 | Geophysical Characterization

The study conducted during the reconnaissance mission focused on the cities of Copiapó and Chañaral (see Figure 1). These cities have populations of 168,831 and 12,345 inhabitants, respectively, accounting for 60.5% of the total population of the Atacama region (Instituto Nacional de Estadísticas (INE) 2024).

Seven ambient-vibration measurements were carried out using two-dimensional arrays with triaxial geophones to derive shear-wave velocity (V_s) profiles at each of them (Chavez-Garcia et al. 2005) and investigate site amplification effects. The spatial distribution of these measurements and the CSN stations used are shown in Figure 5. From these measurements, three were conducted near the CSN stations (VS3, VS4, and VS7), and two (VS5, VS6) were performed in a residential area in Copiapó; all the V_s profiles are presented in the Supporting Information. The survey campaign was complemented with publicly available CSN data from stations A06C, A07C, A08C, and A29C. At the VS5 site, a soil sample was obtained at a depth of 15 cm, which, according to the Unified Soil Classification System (USCS) (ASTM D2487-17 2017), is classified as clayey sand with low plasticity fines (SC-CL). This serves to confirm that the shallow surface sediments at these sites are of alluvial origin, as reported by Arévalo (2005).

In Copiapó, the predominant periods (T_{site}) were determined for each site using the horizontal-to-vertical spectral ratio (HVSr) method (Nogoshi and Igarashi 1971; Nakamura 1989). The processing was performed using the Geopsy software, considering windows between 40 and 60 s of duration, for frequencies between 0.25 and 40 Hz, and applying a Konno–Ohmachi (KO) smoothing filter (Konno and Ohmachi 1998), with a bandwidth coefficient of $b = 40$. Our results were compared with previously published microzonation studies conducted in this zone (Cornejo 2020). Both datasets agree and thus were complemented to produce a map of predominant site period (T_{site}), in which the spatial interpolation was performed using the inverse distance weighting (IDW) method (Watson and Philip 1985). The measurement points in the study area are shown in Figure 6.

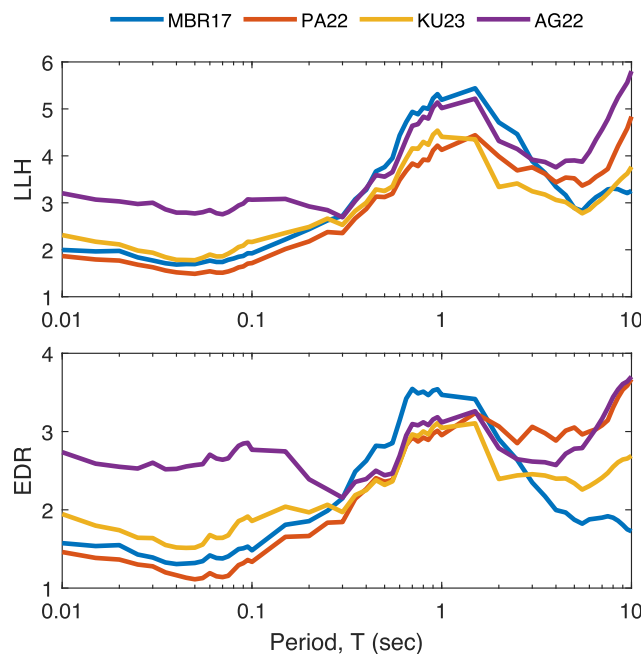


FIGURE 4 | GMM predictability metrics: log-Likelihood (LLH) and Euclidean distance ranking (EDR).

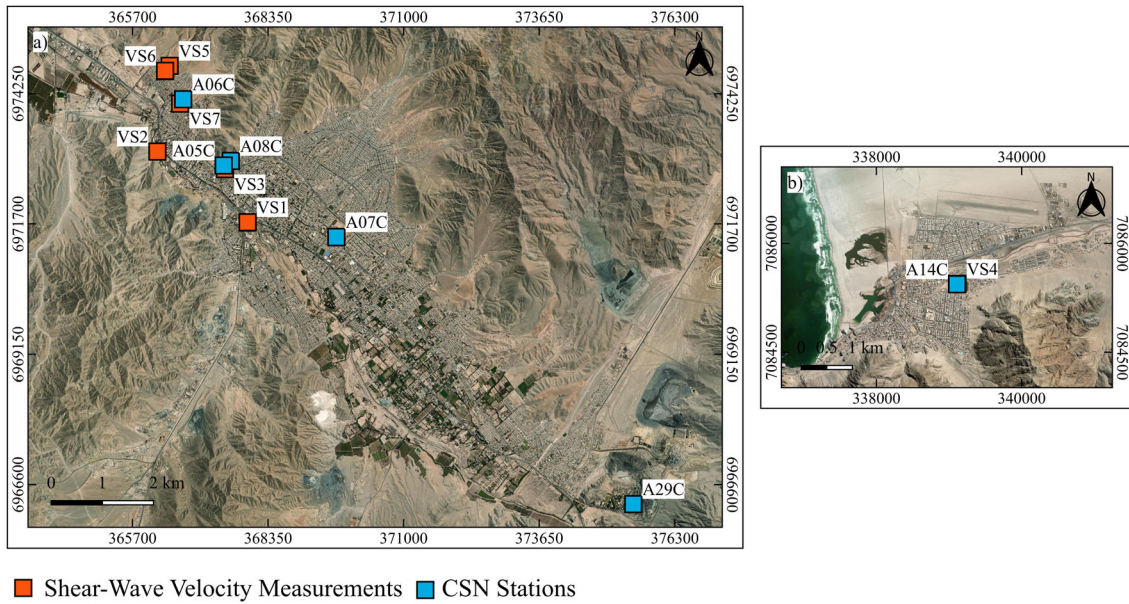


FIGURE 5 | Spatial distribution of shear-wave velocity (V_s) measurement sites and CSN strong-motion stations with collected data: (a) city of Copiapó and (b) city of Chañaral.

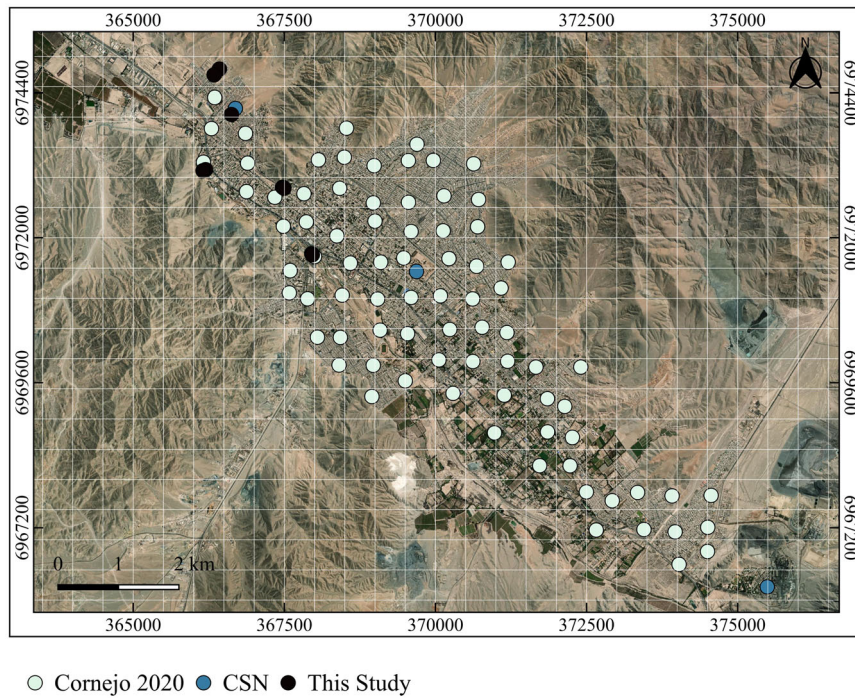


FIGURE 6 | Spatial distribution of measured predominant periods (T_{site}) across the city of Copiapó categorized by data source.

In Chañaral, based on the measurements taken at station A14C, the processed spectral ratios exhibited an ill-defined peak and amplitudes exceeding 4, with periods between 0.68 and 0.73 s (see Figure S6).

Ambient noise measurements were also processed using the spatial autocorrelation (SPAC) technique to determine Rayleigh waves' dispersion curves, which, in turn, were inverted to retrieve the shear-wave velocity (V_s) profiles down to a depth of 100 m (Aki 1957). For measurements that did not exhibit behavior consistent with first-order Bessel functions, the zero-crossing methodology developed by Ekström et al. (2009) was applied to improve the inversion of the dispersion curves. Additional V_s profiles were determined using publicly available data from previously mentioned CSN stations.

Based on the V_s profiles analyzed previously, the harmonic average shear-wave velocity of the upper 30 m (V_{s30}) was computed to assign site classes according to NEHRP (International Code Council 2009) (see Figures S3–S9). In the city of Chañaral, a V_{s30} value of 726 m/s was determined, corresponding to Site Class C (see Figure S6).

Figure 7 shows the resulting V_{s30} values at each measured site, as well as the interpolated map of T_{site} for the city of Copiapó. As observed, most of the measured sites in Copiapó are classified as Site Class C. The highest V_{s30} value (2127 m/s) corresponds to a rocky outcrop, consistent with the measurement location and geomorphological setting. Conversely, the lowest V_{s30} value (288 m/s) was obtained at a site situated in the northern region of the city, where the VS5 measurements was conducted. Particularly, this site is classified as Site Class D and lies within alluvial deposits, in contrast to a nearby site located approximately 100 m away (VS6), which is classified as Site Class C and is located in fluvial deposits (see Figure S10). Despite being closely spaced, from the measurements at the sites VS5 and VS6 reveal distinct V_{s30} values, predominant periods, site classes, and each site is underlain by different geological deposits (Arévalo 2005). These two sites correspond to the locations of the case-study buildings further analyzed in Section 6.

5 | Observed Damage

A total of 16 structures were inspected post-event in Copiapó through on-site observations, photographic records, and georeferencing. A general summary of the characteristics of the assessed structures is reported in Table 1, and their location and some of the identified damages are presented in Figure 8. We chose the structures to inspect by (i) asking local authorities about reported damages, (ii) field team scanning of social media for damage reports, and (iii) driving through the cities of Copiapó and Caldera to catch any evident damage cases. Most buildings were structured in shear walls, except for the Atacama Cultural Center, the MOP building, and the Mallplaza Copiapó Shopping Center, which have mixed systems (shear walls and moment frames). The damage was classified into three categories: D0, no damage; D1, nonstructural damage (broken windows, falling of suspended ceilings, and cracking on partition walls); and D2, structural damage (significant cracking on structural elements, concrete cover spalling, and remanent deformations in shear walls or slabs). In addition, Figure 9 illustrates the spatial relationship between the surveyed structures and the map of the dominant site period (T_{site}).

The evaluation of the buildings revealed that four of them (two bridges and two reinforced concrete [RC] buildings) showed no signs of damage, four exhibited nonstructural damage, and eight presented structural damage. As can be

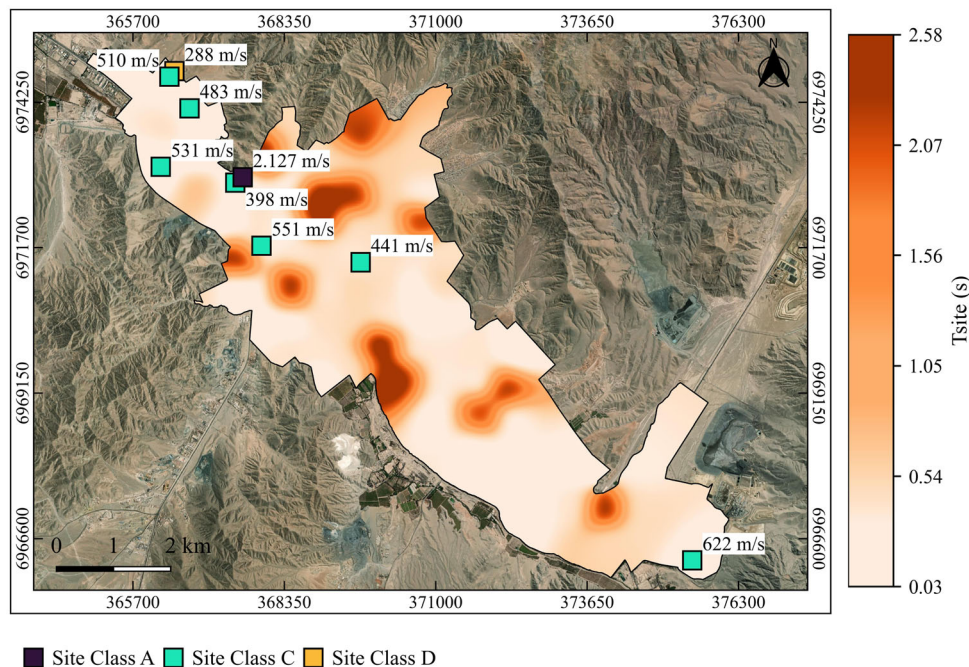


FIGURE 7 | Spatial distribution of V_{s30} , site class, and predominant period in the city of Copiapó. Numbers indicate V_{s30} values from both field measurements and publicly available CSN data; colored squares denote site classification; and the interpolated map illustrates the predominant site period (T_{site}).

TABLE 1 | Inspected structures in Copiapó.

ID	Structure name	Type	Primary material	Use	Damage level
1	Kennedy Bridge	Bridge	Reinforced concrete	Road infrastructure	D0
2	La Paz Bridge	Bridge	Reinforced concrete	Road infrastructure	D0
3	Faculty of Mining	Building	Reinforced concrete	Educational	D2
4	Faculty of Humanities	Building	Reinforced concrete	Educational	D1
5	Municipal Cemetery	Cemetery	Masonry	Heritage	D2
6	Altos de Copiapó II	Building	Reinforced concrete	Residential	D2
7	Altos de Copiapó I	Building	Reinforced concrete	Residential	D0
8	Atacama Cultural Center	Building	Reinforced concrete	Municipal	D1
9	San Francisco Church	Building	Masonry	Heritage	D1
10	MOP (Public Works Ministry)	Building	Reinforced concrete	Government	D0
11	Mallplaza Copiapó Shopping Center	Building	Reinforced concrete	Commercial	D1
12	Historic Center 1	House	Masonry	Commercial	D2
13	Historic Center 2	House	Masonry	Commercial	D2
14	Historic Center 3	House	Masonry	Commercial	D2
15	Near Cemetery 1	House	Quincha	Residential	D2
16	Near Cemetery 2	House	Quincha	Residential	D2

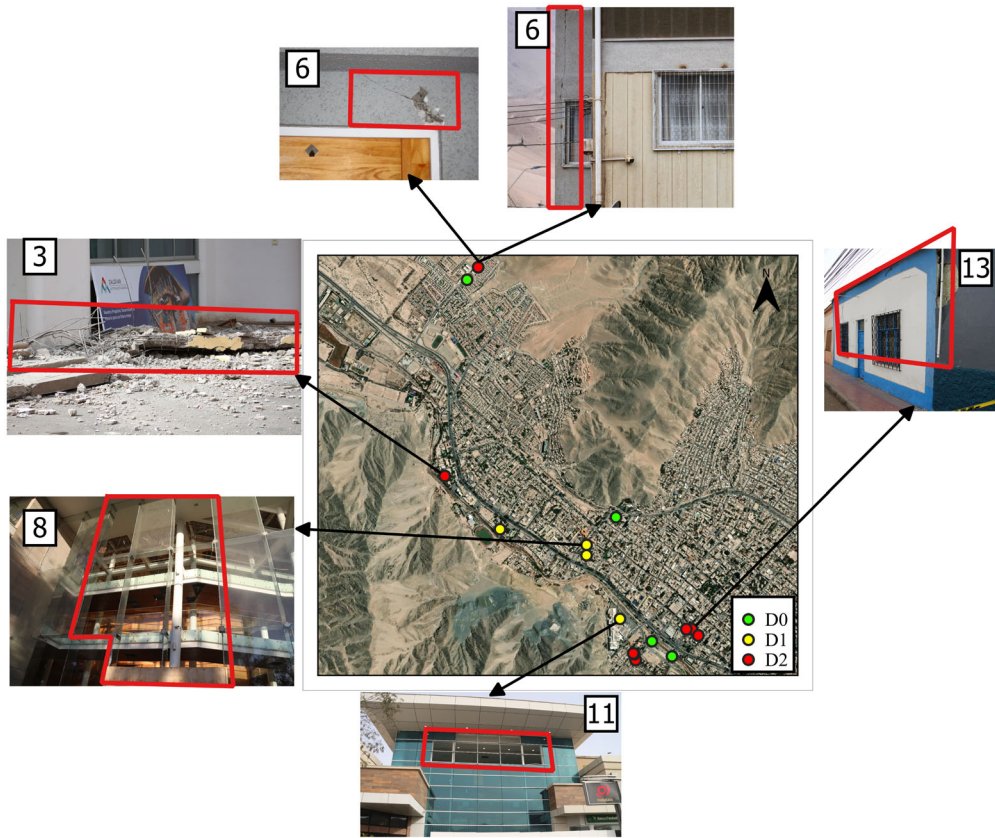


FIGURE 8 | Spatial distribution of inspected structures in Copiapó after the June 6, 2025, earthquake. Each point represents a surveyed building classified according to its observed damage state: no damage (D0, green), nonstructural damage (D1, yellow), and structural damage (D2, red). The photographs illustrate representative examples of the typologies and damage levels recorded across the study area, highlighting façade cracking, detachment of nonstructural elements, and localized structural damage.

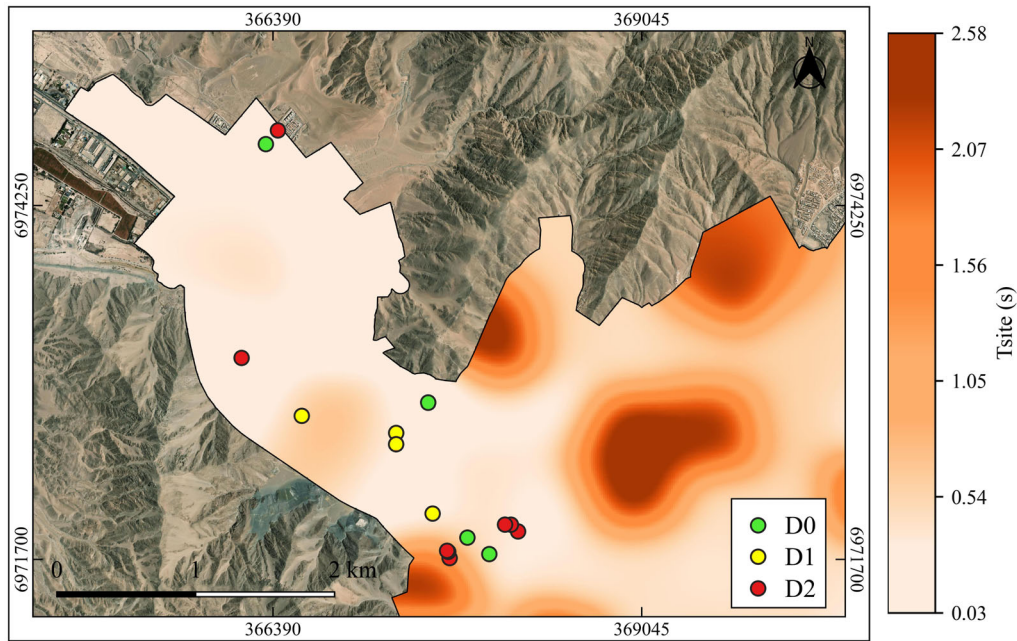


FIGURE 9 | Spatial distribution of surveyed structures within the interpolated map of predominant site period (T_{site}).

observed in Figures 8 and 9, damage states do not seem to follow any spatial distribution pattern. Most inspected RC buildings exhibited mainly nonstructural damage (D1), while most of the structures with damage level D2 correspond to masonry and quincha structures. This pattern of observed damage is consistent with expectations, given that masonry and nonengineered construction systems are usually more vulnerable than RC because of their intrinsic material brittleness and the lack of maintenance of these structures (D'Ayala and Benzoni 2012; Cuadra et al. 2012; Astroza et al. 2012).

The absence of a clear city-scale spatial pattern is consistent with two factors: (1) the heterogeneous structural typology of the inspected sample, in which vulnerability is the dominant source of variation in damage, and (2) the moderate magnitude of the event, which did not push most low- and mid-vulnerability structures into demand levels at which site amplification effects would become decisive. For instance, the 2010 Mw 8.8 Maule earthquake produced a damage pattern of highly vulnerable buildings that can be correlated with predominant site period and bedrock depth, while low- and mid-vulnerability buildings showed damage largely independent of site conditions (Montalva et al. 2016). Therefore, a larger event in the Atacama region would be expected to produce a more detectable city-scale spatial pattern, particularly in the older masonry and quincha typologies concentrated in the historic center.

5.1 | Nonstructural Damage

Significant nonstructural damage was observed in modern public-use buildings, including facilities such as the Mallplaza Copiapó shopping center, the Atacama Cultural Center, the Faculties of Humanities and Mines at Atacama University (UDA), and the Teletón Foundation Therapy Center at Copiapó. The nonstructural damage registered in these buildings included broken window glass, detachment of glass façades, falling of suspended ceilings, and cracking of partition walls.

Figure 10 shows the Faculty of Mines at Atacama University, a two-story building, where both structural and nonstructural damages were observed. The damage was concentrated on the second floor and the roof. Structural damage was mainly identified in the concrete cladding, columns, and slabs, while secondary nonstructural elements, such as windows, also exhibited damage.

Figure 11 shows the post-event damage at the Teletón Institute at Copiapó, a one-story high RC building therapy center. This building exhibited nonstructural damage, where evacuation routes and public areas, including therapy rooms, were obstructed due to the detachment of suspended ceilings, recessed ceiling lights, and other hanging nonstructural elements.

Figures 12 and 13 show damage to the façade of the Atacama Cultural Center and the Mallplaza Copiapó shopping center, respectively. The first is a three-story building structured with RC shear walls with steel frames, and the second is a structure composed of different blocks with varying heights from three to six stories and mainly structured with RC

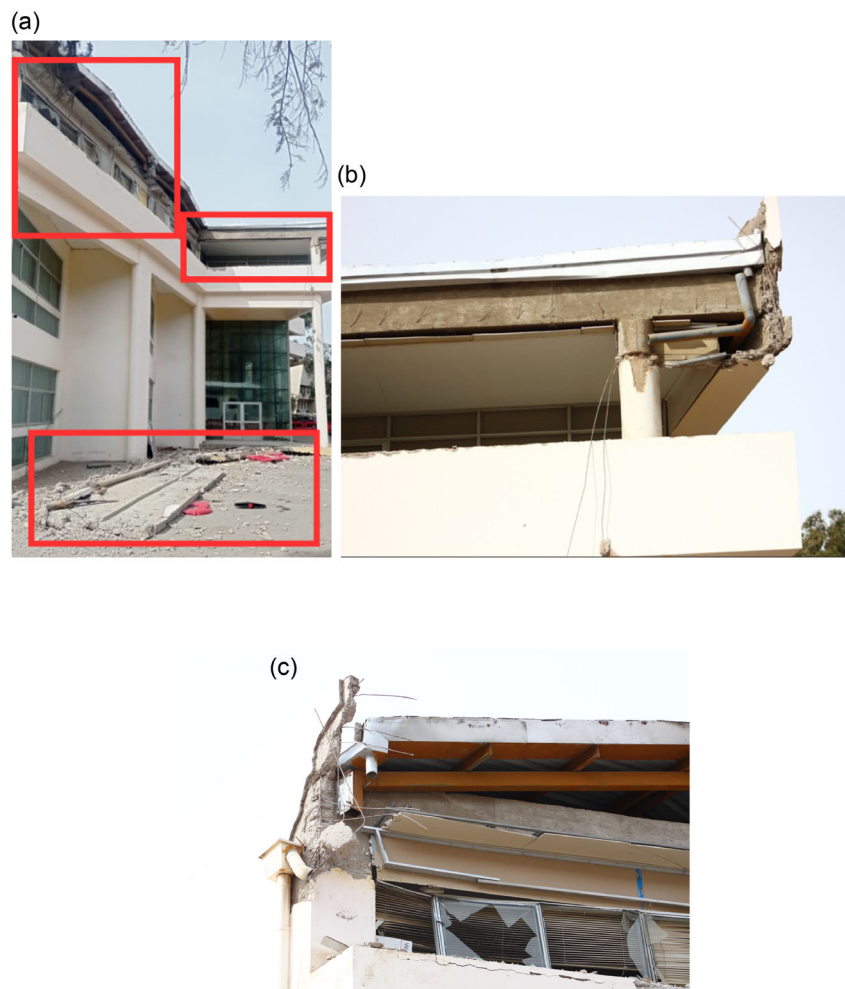


FIGURE 10 | Nonstructural and structural damage observed at the Mines Department of the University of Atacama: (a) debris and material detachment on the building exterior, (b) localized damage with concrete spalling along the roof slab edge, and (c) damage to the roof edge of the building.

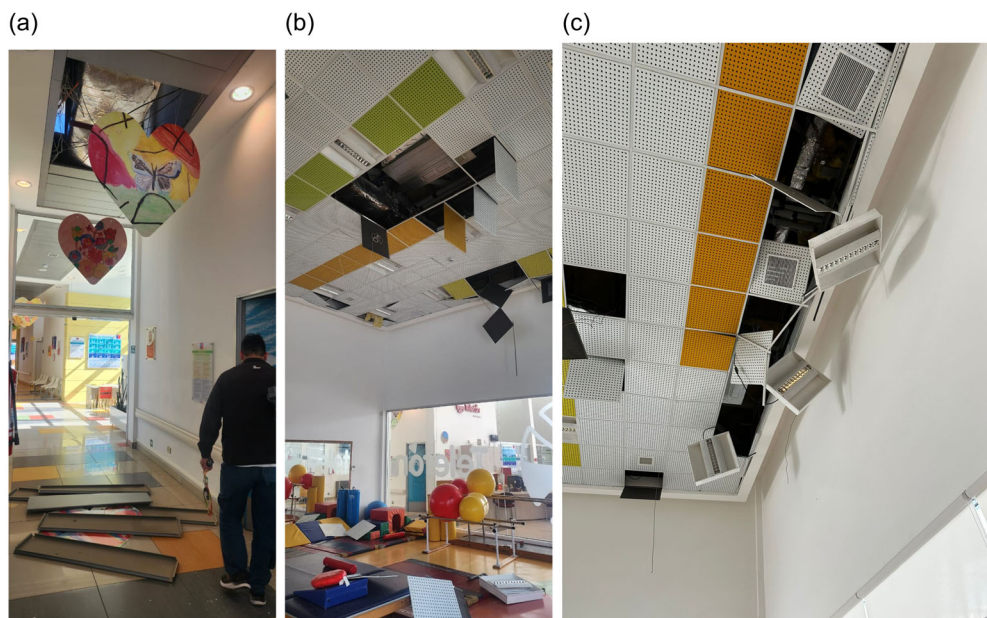


FIGURE 11 | Representative examples of nonstructural damage observed in the Teletón Foundation facility: (a) blockage caused by the detachment of nonstructural elements in evacuation zones, (b) damage in therapeutic areas, and (c) ceiling detachment.

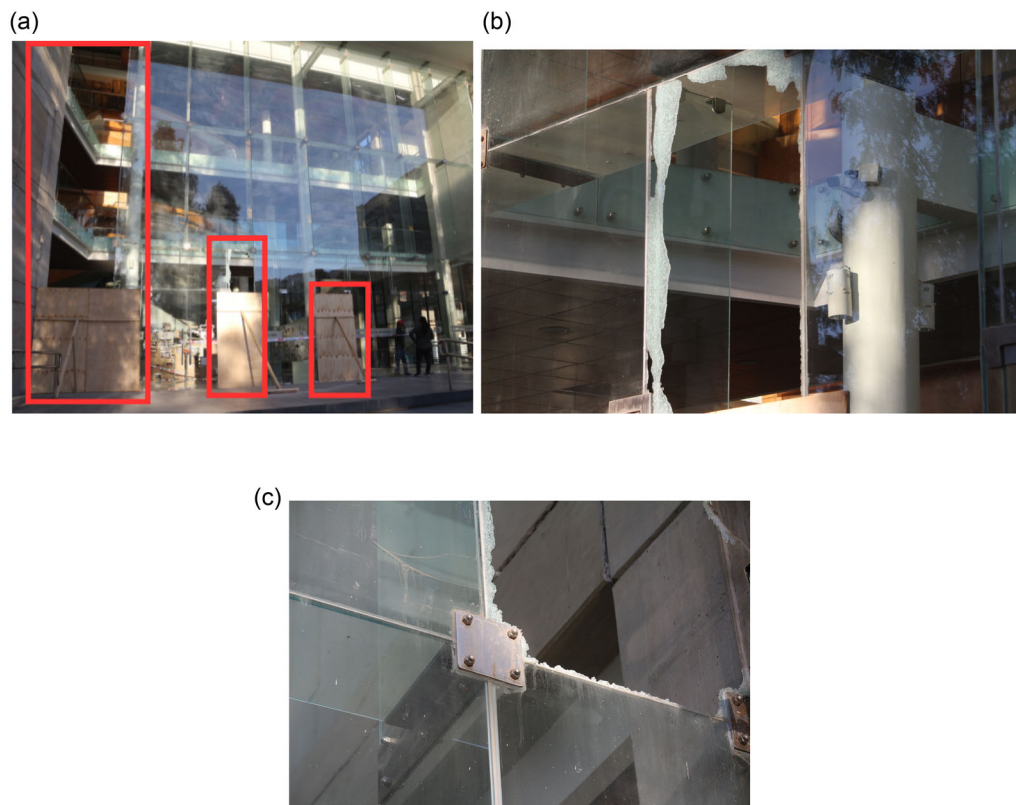


FIGURE 12 | Nonstructural damage to glass wall systems in the cultural center: (a) detachment of glass panels, (b) cracking and partial failure of glass panels, and (c) localized glass breakage near connection points in the cultural center.

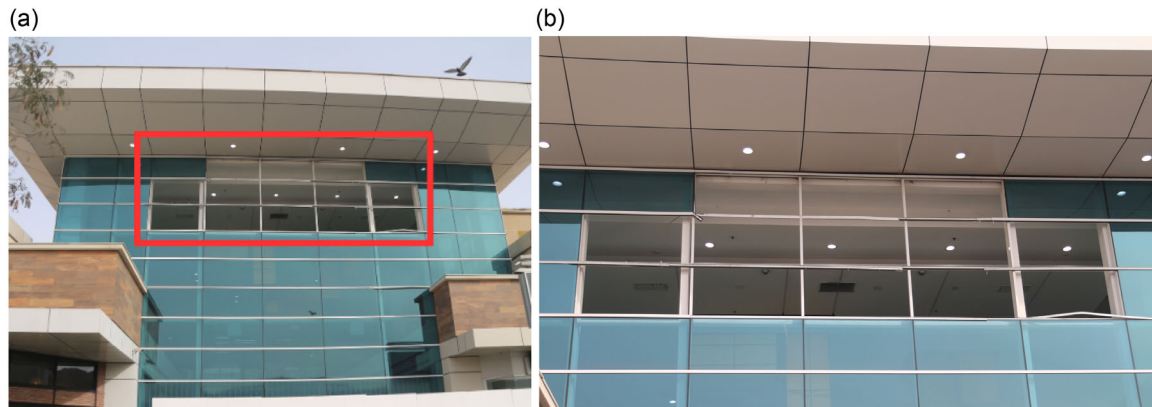


FIGURE 13 | Representative examples of nonstructural damage to glass panel systems: (a) detachment of glass panels and (b) close-up view of the failure in areas exposed to public evacuation in the shopping center.

moment frames. Both buildings exhibited breakage and falling of glass panels, which resulted in the scattering of shards across public areas, including emergency exits. These examples of nonstructural damage highlight the impact that they can cause to the general population: inhabitation of therapy rooms and public areas and blocking of evacuation paths, restricted by Article 4.2 of [Ministerio de Vivienda y Urbanismo \(1992\)](#).

Given the moderate magnitude (M_w 6.4) of the Copiapó inslab earthquake for the Chilean context, the extent of the damage, which impeded immediate occupancy, indicates an unacceptable level of performance considering the critical function and the high level of daily attendance of these facilities. This underscores the importance of reviewing the adequacy and implementation of nonstructural seismic provisions, whose failure may lead to injuries, significant economic losses, and partial or complete loss of functionality ([Miranda et al. 2012](#)).

5.2 | Structural Damage

Significant structural damage was observed in quincha structures (one-story houses and aggregates), which are exemplified in Figure 14. Some walls presented diagonal cracking characteristic of shear failures (Figure 14a). Other walls presented out-of-plane disconnection from transversal and confining elements (Figure 14b,c). There was also evidence of plaster spalling and deterioration of the subadjacent structural element on significant portions of walls (Figure 14d), in some cases even resulting in partial collapse of walls. Postseismic damage might be expected in buildings of this material because they are mostly old structures that were not engineered and do not comply with current earthquake-resistant code requirements.

A housing complex, Altos de Copiapó II, composed of five 4-story RC apartment buildings presented structural damage. The buildings showed extensive cracking between structural elements, as shown in Figure 15. The main damage patterns include exposed reinforcement within the walls, uneven floor levels in several apartments, and separation between structural and nonstructural elements (such as partition walls), as well as misaligned stairs. These buildings are relatively new (they were built in 2015), implying that they should not have experienced damage in earthquakes like the assessed one. For this reason, a thorough assessment of these buildings was carried out.

6 | Case Study: Engineered Residential Complexes

Special attention was given to two RC housing complexes, Altos de Copiapó I and Altos de Copiapó II, located no more than 100 m apart. The two complexes were constructed from the same set of structural drawings and share identical typology, plan dimensions, structural system, and nominal material specifications. Each complex is composed of four and five 4-story shear-wall RC apartment buildings. Within each complex, paired RC buildings are linked by an

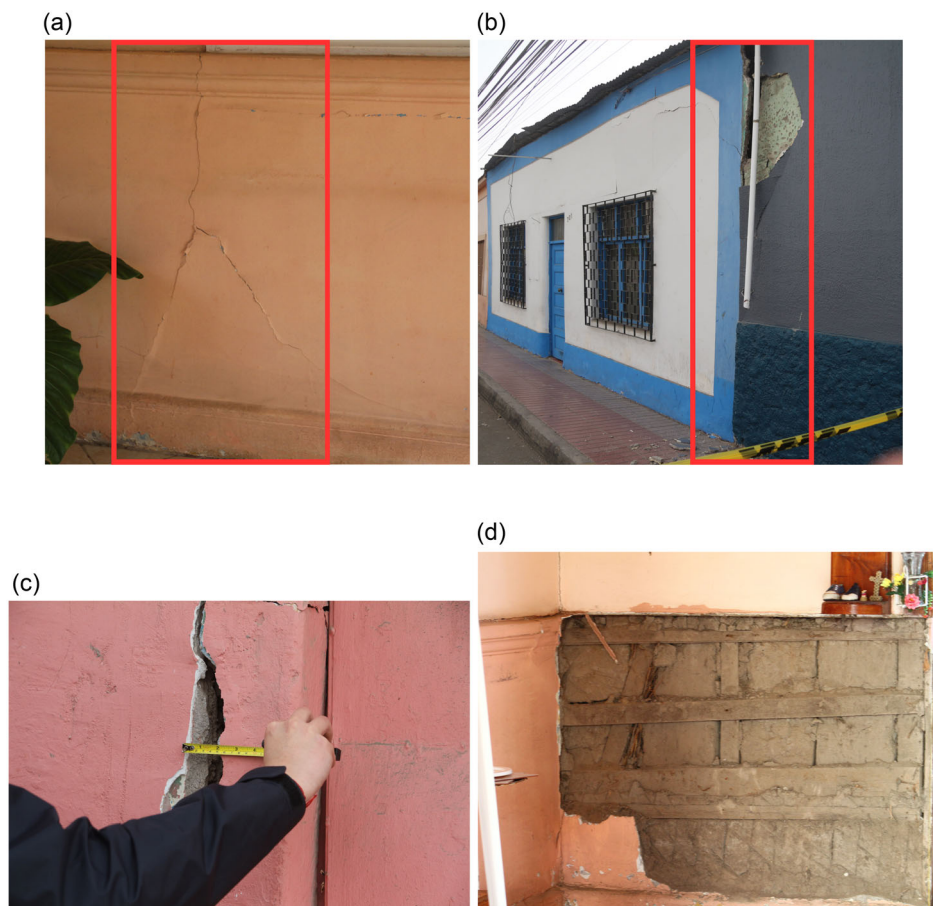


FIGURE 14 | Representative structural damage observed in quincha structures: (a) diagonal cracking in infill walls, (b) separation and slipping of the wall relative to the structural frame, (c) severe cracking and material detachment, and (d) partial loss of wall material exposing the internal structure.

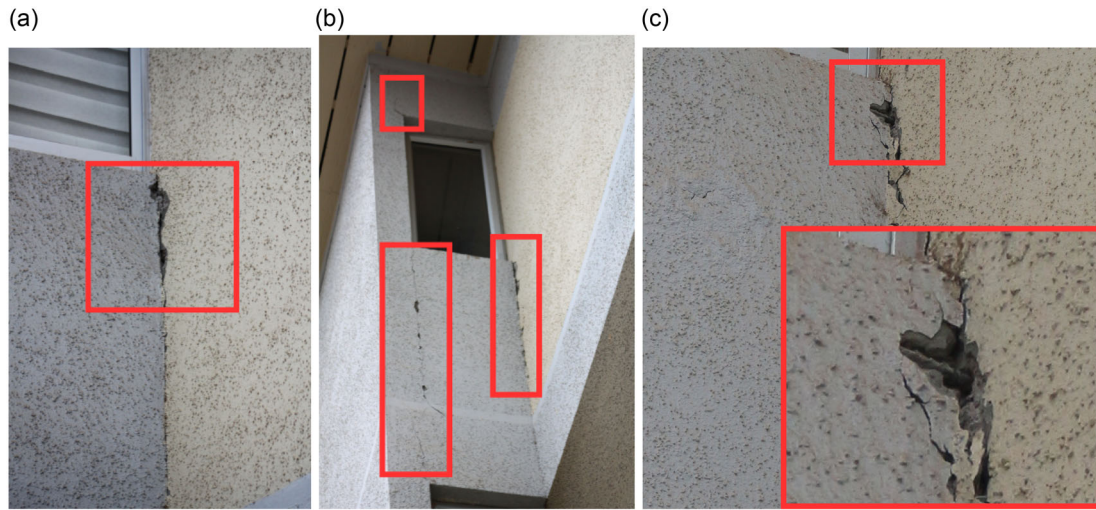


FIGURE 15 | Structural damage observed between walls in the Altos de Copiapó II building complex, where cracking of the reinforced concrete led to the exposure of the reinforcement.

intermediate metallic stair core; the two buildings and the stair core are three structurally independent units, with each building-stair interface bridged only by a nonstructural cover strip resting on top of the RC slab. Although all buildings shared the same typology, materials, and design and were no more than 100 m apart, they exhibited a notably different behavior: The Altos de Copiapó II complex suffered structural damage, while the buildings of Altos de Copiapó I remained undamaged. The case study, therefore, isolates structural typology, the dominant source of variation in the city-scale damage survey, and provides a setting in which the effect of local site conditions can be examined.

We estimate the fundamental translational periods in the principal direction (Y, short direction of buildings) and viscous damping ratios for the nine buildings within these residential complexes. The buildings were tagged as 6.1–6.5 and 7.1–7.4, respectively, using the building notation from Table 1), as shown in Figure 16. All assessed buildings were built

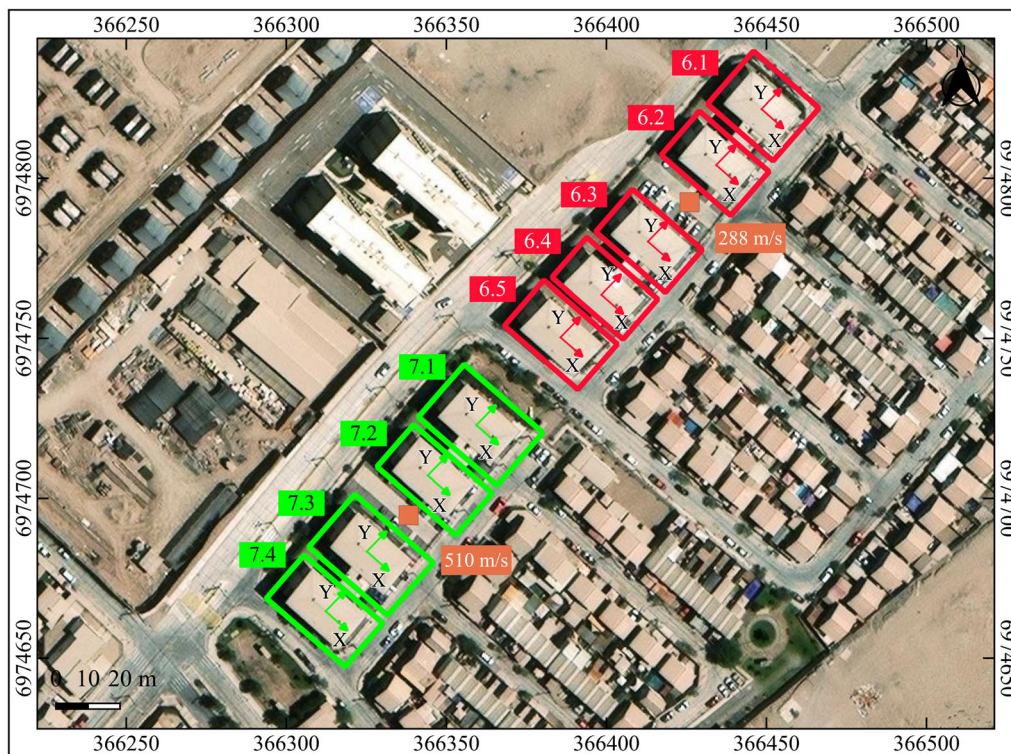


FIGURE 16 | Spatial distribution of surveyed buildings across the study area. Condition classes are distinguished by color: undamaged (7.1–7.4, Altos de Copiapó I) in green and damaged (6.1–6.5, Altos de Copiapó II) in red. Building notation follows Table 1. The on-site V_{s30} measurements are shown in orange.

according to the same design conditions and have a plan of 22 m by 14 m and are approximately 15 m high. The analysis addresses two practical questions: whether post-event damage documented on site only in Altos de Copiapó II buildings manifests as systematic period elongation and whether damping inferred from ambient vibrations shows patterns compatible with increased frictional/viscous dissipation in damaged systems.

At each building, triaxial seismometers were installed at the geometric center of the fourth floor. Two sensor configurations were used: (i) a two-sensor layout, aligned with the Y-axis centerline on the fourth floor and separated by 22 m—applied to buildings 6.4 and 7.3 (see Figure 16), and (ii) a single-sensor layout on the fourth floor (near the geometric center of the story) for all remaining buildings. Figure 17 shows a plan view of the buildings and the sensor locations. The two-sensor layout permits direct disambiguation of the first translational and torsional modes: A translational mode produces an in-phase response at the fundamental frequency, whereas a torsional mode produces a 180° phase difference between the two sensors. Figure S11 shows the coherence and phase between the two sensors deployed in building 6.4; the coherence peak coincides with a phase lag near 0° , indicating a translational mode. For the buildings instrumented with a single sensor close to the geometric center, we adopt the same identification based on the following: (i) All nine buildings share the same nominally symmetric plan and shear-wall layout (Figure 17), placing the center of stiffness near the center of mass and minimizing rotational coupling of the first mode and (ii) the two-sensor measurements verify this expectation directly.

Raw accelerations were linearly detrended and band-pass filtered between 0.20 and 40 Hz (fourth-order, zero-phase) to reduce low-frequency drift and high-frequency noise. Quality assurance combined the following: (i) short-time average over long-time average (STA/LTA) outlier rejection on raw traces and on a low-pass reference (short/long windows = 1 s/30 s) as in Allen (1978) and the comparative study of Withers et al. (1998), (ii) robust first-difference “glitch” detection with MAD-scaled thresholds and temporal clustering, (iii) step detection via robust running means with padding to avoid filter start-up effects, and (iv) mask dilation by 8 s to suppress edge leakage. Records were segmented into 120 s windows with 50% overlap. A window was retained if $\geq 99.5\%$ of samples passed QA and ≤ 3 s were flagged, balancing frequency resolution and statistical stability against sparse transients.

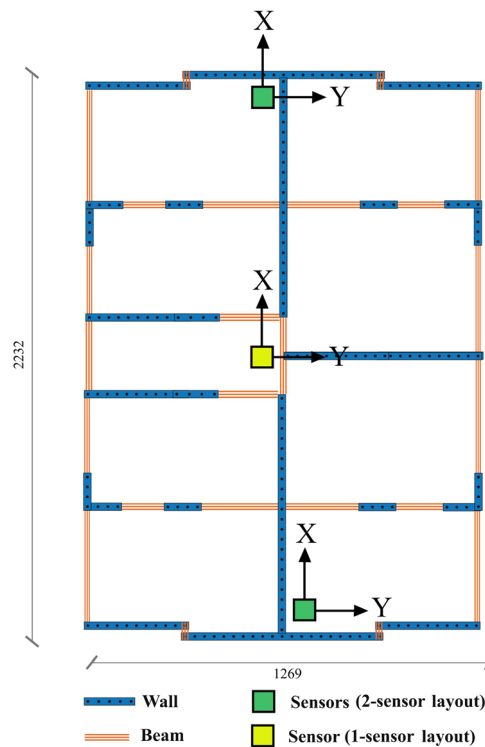


FIGURE 17 | Plan view of the buildings with the location and orientation of the sensors used to estimate the modal properties of the case-study buildings. All nine surveyed buildings share the same design. Sensor configurations are indicated in two colors: in green, the two-sensor layout applied to buildings 6.4 and 7.3, with both sensors lying on the Y-axis centerline of the fourth floor and separated by approximately 22 m; in yellow, the single-sensor layout adopted for the remaining buildings, positioned close to the geometric center of the fourth floor. Dimensions in centimeters.

For each accepted window, one-sided power spectral densities (PSDs) were computed using Welch's method (20 s Hann, 50% overlap) based on Welch (1967), mapped onto a log-spaced grid from 1 to 30 Hz (100 points), and smoothed with the KO kernel (Konno and Ohmachi 1998) with bandwidth coefficient $b=70$. The predominant frequency f_0 was sought in 4–10 Hz—consistent with midrise RC buildings—by maximizing the KO-smoothed median in that band and refining the pick via a local parabolic fit in log-PSD space. Reliability was improved with a two-pass signal-to-noise ratio (SNR) gate: first, a preliminary f_0 , and second, per-window SNR at f_0 as the ratio of the peak ordinate to the median PSD within a ± 0.5 Hz sideband, excluding ± 0.08 Hz around the peak. Windows with an SNR ≥ 8 dB were kept. The median spectrum and f_0 were then recomputed from gated windows. This is consistent with ambient-vibration and spectral-ratio practice documented by Ktenidou et al. (2011) and with building-scale ambient-vibration analyses such as those by Michel et al. (2009).

Damping ratios were estimated with the random decrement (RDT) method in the sense of Ibrahim (1977). Accepted windows were concatenated with guard intervals to avoid edge effects and then narrow-band filtered around f_0 using a 2% fractional bandwidth (not less than 0.2 Hz). Zero-crossing uptriggers (threshold 0σ) produced an RDT signature from at least 800 shots. The natural logarithm of the envelope of this signature was regressed over a fixed decay segment of 1.0–2.2 s to obtain the viscous damping ratio.

The range of measured fundamental periods (0.136–0.23 s) is coherent with the values reported for Chilean RC buildings by Lagos et al. (2021). Additionally, across the sample, a consistent association is observed between the identified dynamic properties and the documented damage state, as shown in Figure 18. Buildings classified as damaged (D2) exhibit systematically longer fundamental periods in the Y direction and higher RDT-based viscous damping ratios than their undamaged (D0) counterparts. The median fundamental period increases from 0.152 s (undamaged, D0) to 0.191 s (structural damage, D2), an increase of approximately 20%, while the median damping ratio increases from 1.72% to 1.97%, an increase of about 10%. Although the numerical ranges overlap, the ordering is consistent across buildings.

It is worth noting that the observed period elongation in the damaged complex may reflect not only structural stiffness degradation associated with cracking and damage but also differences in site conditions and soil–foundation–structure interaction (SFSI) effects, given that the Altos de Copiapó II complex is located on softer soil ($V_{s30} = 288$ m/s, Site Class D) than its undamaged counterpart. Similar observations have been reported for masonry buildings founded on soft soils in Italy (de Silva and Silvestri 2025). This suggests that local soil conditions may have contributed to the observed differences in dynamic response. However, since the measurements were performed after the earthquake, the observed period elongation likely reflects the combined influence of SFSI effects and structural stiffness degradation. A second caveat concerns modal identification: For the seven buildings instrumented with a single sensor, the recordings alone cannot

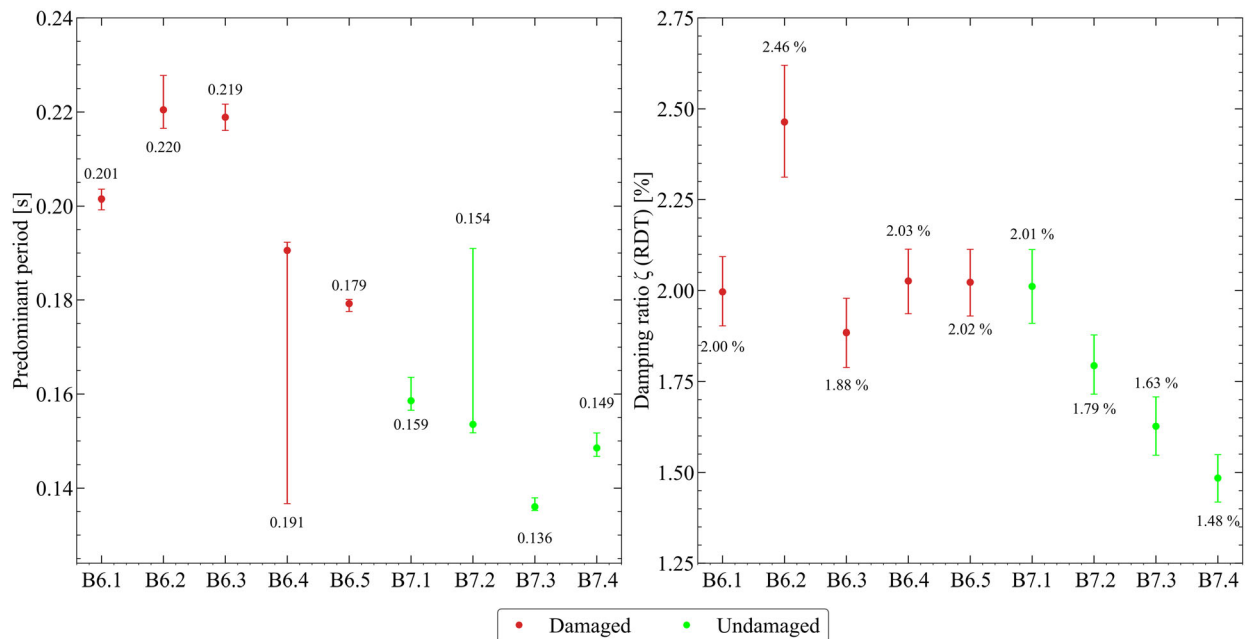


FIGURE 18 | Predominant period in the principal (Y) direction (left panel) and corresponding viscous damping ratio (right panel) for all case-study buildings. Color coding denotes observed damage state from the field survey: Buildings with structural damage (6.1–6.5) are shown in red, whereas undamaged buildings (7.1–7.4) are shown in green. Error bars show the range of observed values for each building among time windows of samples.

rule out a torsional contribution, and the translational identification rests on the plan-symmetry argument and the two-sensor verification previously described. Pre-earthquake measurements of these buildings are not available, so the inference that the observed period elongation and damping increase are damage-induced relies on indirect evidence. Two features of the data support this interpretation against the alternative hypothesis of pre-existing structural variability. First, the case study comprises nine nominally identical buildings (five damaged, four undamaged); the period and damping values cluster systematically by damage state, with damaged buildings having higher median period and damping than the undamaged group. Pre-existing variability among nominally identical structures would generally produce unstructured scatter rather than the observed between-group separation. Second, both period and damping shift jointly in the directions expected from concrete cracking (i.e., reduced effective stiffness and increased hysteretic dissipation), and this joint shift aligns with the independently observed visual damage state. A pre-earthquake baseline would nonetheless be required to confirm this interpretation directly, and we acknowledge this limitation explicitly.

Note that the predominant periods measured are 0.29 and 0.25 s for the sites of buildings B6 and B7, respectively, similar to the fundamental periods measured in the buildings. This observation cannot explain the observed damage, given the slight differences observed; however, the possible effects of resonance between the structures and the soil cannot be completely ruled out.

A spatial association along the block is also apparent: The sector where buildings with structural damage cluster aligns with the upper end of the period and damping ratio distributions, whereas the undamaged sector aligns with the lower end. Adjacent buildings tend to exhibit similar values, suggesting an along-block gradient (from southwest to northeast) in both period and damping, which is consistent with the damage pattern. While these effects clustered damage could have been affected by SFSI effects, the strong difference in V_{S30} values is likely more significant because it would have caused markedly different ground-motion intensities at both locations. Moreover, the complex Altos de Copiapó II, with its five buildings with structural damage, is located at the site with the lowest measured V_{S30} value. These observations underscore the importance of a proper assessment of site characteristics, given that local amplification effects may become highly relevant.

7 | Conclusions

The Mw 6.4 Copiapó inslab earthquake of June 6, 2025, occurred within the Atacama seismic gap, where this moderate magnitude event produced strong ground-motion intensities and widespread nonstructural damage, providing a unique opportunity for a multidisciplinary field survey to take lessons from its effects.

Twenty-three strong-motion stations recorded the event, with the highest PGA of 0.44 g and a highest PGV of 14.5 cm/s. The observed peak ground values were systematically higher than those predicted by available subduction-zone ground-motion models.

The geophysical characterization carried out in Copiapó and Chañaral revealed spatial variability in subsurface conditions, particularly in the measured V_{S30} values, predominant periods, and site classifications. The integration of SPAC-derived dispersion curves and publicly available CSN data allowed us to describe local site conditions across both cities. Notably, sites located only a few tens of meters apart, such as VS5 and VS6, exhibit large differences in their geophysical properties, which agrees with the available information of geological deposits. This variability has implications for site effects that may be considered in seismic risk assessment.

Topographic effects were not explicitly modeled in this study. The measurement sites are situated within the flat urban zone of Copiapó, an alluvial basin with gentle topographic gradients (mean elevation 369.6 m a.s.l., standard deviation approximately 8 m), where significant two-dimensional or three-dimensional topographic amplification is unlikely. Under these conditions, site response is primarily governed by the near-surface sedimentary stratigraphy rather than by topographic irregularities. Future research incorporating deeper geophysical characterization of the basin structure could further refine these estimates, especially near the basin margins, where topographic and basin-edge effects warrant further evaluation.

The field survey of 16 structures showed a mixed damage pattern within this sample of damaged structures in the city of Copiapó. The assessed older masonry and quincha buildings exhibited severe structural damage (D2). From the inspected RC buildings, most of them presented nonstructural damage such as cracking and façade detachment, although a residential complex (Altos de Copiapó II) of five buildings and structured in shear walls exhibited structural damage. Particularly concerning was the functional disruption observed in public and essential facilities—including universities, cultural centers, and rehabilitation centers—where nonstructural components compromised evacuation routes and

service continuity. Overall, the nonstructural damage in public spaces and evacuation zones highlights the need to review the design and implementation of nonstructural components in public buildings.

The case study of the Altos de Copiapó I and II residential complexes revealed contrasting structural performance despite nearly identical design, construction, and location. Ambient-vibration analyses confirmed longer predominant periods and higher viscous damping ratios in the buildings with structural damage, suggesting stiffness degradation and increased energy dissipation.

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Conflicts of Interest

The authors declare no conflicts of interest.

Data Availability Statement

Strong ground motions were obtained from the National Seismological Center of Chile (Centro Sismológico Nacional [CSN]) via their open-access website (<https://evtdb.csn.uchile.cl/>, last accessed November 2025). Seismic data processing and calculations were performed using MATLAB R2024b (MATLAB 2024). The map visualizations in Figure 1 benefited from the Generic Mapping Tools (Wessel et al. 2019) implemented in the PyGMT library (Uieda et al. 2021). The raw data and metadata for the ambient-vibration measurements used in this work are available at <https://doi.org/10.5281/zenodo.18233849> (Montalva et al. 2026).

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Supporting Information

Additional supporting information can be found online in the Supporting Information section. **Figure S1:** Three-component ground-motion records sorted by station latitude, shown as a function of time since the earthquake origin. Each trace has its station code indicated at the end of its time history. **Figure S2:** Zoom-in on the nearest stations to the earthquake, shown in Figure S1. The three-component ground-motion records are sorted by hypocentral distance. Each trace indicates its station code at the end of each time history. **Figure S3:** (a) Shear-wave velocity profile of the upper 30 m at site VS1. (b) Horizontal-to-vertical spectral ratio (HVSr) at site VS1. **Figure S4:** (a) Shear-wave velocity profile of the upper 30 m at site VS2. (b) Horizontal-to-vertical spectral ratio (HVSr) at site VS2. **Figure S5:** (a) Shear-wave velocity profile of the upper 30 m at site VS3. (b) Horizontal-to-vertical spectral ratio (HVSr) at site VS3. **Figure S6:** (a) Shear-wave velocity profile of the upper 30 m at site VS4. (b) Horizontal-to-vertical spectral ratio (HVSr) at site VS4. **Figure S7:** (a) Shear-wave velocity profile of the upper 30 m at site VS5. (b) Horizontal-to-vertical spectral ratio (HVSr) at site VS5. **Figure S8:** (a) Shear-wave velocity profile of the upper 30 m at site VS6. (b) Horizontal-to-vertical spectral ratio (HVSr) at site VS6. **Figure S9:** (a) Shear-wave velocity profile of the upper 30 m at site VS7. (b) Horizontal-to-vertical spectral ratio (HVSr) at site VS7. **Figure S10:** Geological map of the city of Copiapó and the spatial distribution of the harmonic average shear wave velocity of the upper 30 m (V_{S30}) at the two sites in the case study. **Figure S11:** Coherence (top) and phase lag (bottom) between the two sensors deployed on the fourth floor of building 6.4, aligned along the Y-axis centerline and separated by 22 m. The dotted vertical line marks the identified fundamental frequency $f_0 \approx 5.2$ Hz, at which the coherence peaks and the phase lag is close to 0° , identifying the first mode as translational. **Table S1:** Inferred stratigraphic profile at site VS1. **Table S2:** Inferred stratigraphic profile at site VS2. **Table S3:** Inferred stratigraphic profile at site VS3. **Table S4:** Inferred stratigraphic profile at site VS4. **Table S5:** Inferred stratigraphic profile at site VS5. **Table S6:** Inferred stratigraphic profile at site VS6. **Table S7:** Inferred stratigraphic profile at site VS7.