Bulletin of Earthquake Engineering
https://doi.org/10.1007/s10518-022-01453-5

CASE STUDY REPORTS

Seismic performance validation for RC building structures damaged by Durres earthquake, Mw6.4, 26 November 2019, Albania

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Received: 20 December 2021 / Accepted: 7 June 2022

Abstract
The earthquake that took place at 03:56 local time on November 26, 2019, with Mw = 6.4, struck the west part of Albania and caused heavy damages to many public and residential buildings in the districts of Durres, Tirana, Lezha, Shkodra, Diver, Berat and the surrounding areas. Immediately after the earthquake, teams from the Institute of Earthquake Engineering and Engineering Seismology arrived and made rapid visual assessment of 169 damaged buildings in the affected area. During the inspection, severe damages to structural and nonstructural elements were found. This paper shows the analytical proof of the seismic behaviour of the structures in which damages were observed during visual assessment. Elastic (static and equivalent seismic force) analyses of elements were performed up to ultimate state of strength along with dynamic analyses of low-, mid- and high-rise buildings with different structural systems and characteristics corresponding to the period of their construction and the design legislation for such type of structures in Albania. The ductility and displacement capacities were defined for each of these groups of structures, while dynamic analysis was performed to define the displacements and the ductility caused by the real earthquake record and the intensity of the earthquake recorded in Durres. The results from the analyses confirm the behaviour of low- and mid-rise structures under this earthquake considering that damages observed on-site completely correspond to damages that occur under the displacements obtained from the analytical investigations. Since high-rise buildings were not found in the stricken region, their behaviour under an earthquake with the same characteristics as the one that took place in November 2019 was predicted applying the same methodology used for the low- and mid- rise buildings.

Keywords Durres earthquake · Seismic performance · Displacement capacity

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Published online: 24 June 2022

1 Introduction

Earthquakes are natural phenomena that release large amounts of energy and result in structural damage, financial loss and, in the worst case, human casualties. Earthquakes occur less frequently than other natural hazards, but account for the largest proportion of losses. Using modern regulations and codes, structures are designed in a way that decreases the seismic risk, but each new strong earthquake inflicting damage to structures and loss of human lives, is an indicator and reminder of the missteps made during construction of buildings. During recent worldwide earthquakes, such as the 1994 Northridge earthquake in the United States (US), the 1995 Kobe earthquake in Japan, the 1999 Chi-Chi earthquake in Taiwan, and the 2008 Wenchuan earthquake in China as well as earthquakes that took place in the Balkan Region, such as Montenegro 1979, (Yugoslavia), Bitola 1994, (N. Macedonia), Petrinje, 2020 (Croatia), Durres 2019 (Albania), Laquila, 2016, (Italy), many reinforced concrete (RC) frame structures experienced substantial damage.

The existing seismic design methods allow structures to undergo plastic deformations under large earthquakes, while remaining elastic under small or moderate earthquakes. The plastic deformation enables dissipation of earthquake energy and is intended to prevent structural collapse. While this design method is highly effective for protecting human lives, it does not fully account for people's lives after the earthquakes (Takagi and Wada 2019). Many times, when buildings are not designed or constructed following the code, they get damaged during small earthquakes, or collapse under large earthquakes. The cause of these unwanted events is usually poor construction, man-made mistakes and poor concrete quality (Binici 2007; Çağatay 2005). The critical damage to important architectural achievements in the recognizable city centre of Zagreb was caused by the severe earthquake that hit Zagreb on March 22, 2020. Post-earthquake assessment, damage classification and failure patterns of residential buildings prove that awareness and preparedness are vital to prevent catastrophic consequences of an earthquake and enable on-time response after an earthquake (Stepinac et al., 2020). The earthquake with Mw6.4 that took place in Durres on 26th November 2019 caused extensive damage to a large number of structures with different structural systems, different number of storeys and different purpose. Following the Durres earthquake, responding to the call of the Government of N. Macedonia (29.11.2019), the Institute of Earthquake Engineering and Engineering Seismology (IZIIS) in Skopje sent three teams composed of 9 experts to the affected area with a reconnaissance mission. The rapid damage assessment mission was realized in the period 2–13.12.2019, with the support of local engineers, resulting in 169 inspected structures in the territory of Durres and Shijak. The inspection showed that the extensive damage found in structural and non-structural elements was the result of inconsistent application of recent knowledge in design, construction and quality control of earthquake resistant structures. Structural flaws were observed in design and construction as well as inappropriate quality of built-in materials (Sheshov et al. 2021), including lack of transverse ties, plastic hinges in columns, placement of columns with a single orientation, soft and weak story phenomenon, short column problem, asymmetrical buildings, construction mistakes, etc. Freddi et al. (2021) stated that the 26th November earthquake in Albania highlighted the deadly connection between the ineffective law enforcement in the construction process and the high seismic vulnerability due to the presence of unauthorized structural interventions (e.g., the construction of additional floors or column removal at the ground floor).

Generally, most building structures in Albania and Balkan normally represent low- to medium-rise RC frames. Only limited studies Zhulegu and Bilgin (2012), Hysenlliu and

Bilgin (2020) have been conducted to investigate the seismic performance of typical RC building frames designed on the basis of the Albanian Code. Also, studies for the performance assessment of RC buildings normally use methods of analysis recommended in different codes, which include push over analysis and nonlinear static analysis procedures (Kueht and Hueste 2009; Panagiotakos and Fardis 2004; Kotronis et al. 2005; Li et al. 2007). The objective of this study was to investigate the seismic performance of typical RC frame structures built in the Balkan region by performing nonlinear dynamic analysis and validate their performance by the reported observed damages inflicted by the 26th November earthquake to low-, mid- and high-rise buildings with different structural systems and characteristics, built according to the design codes in Albania. The results are presented in terms of key response parameters, including inter-story drift, displacement and ductility demands. Based on the observed and analyzed damages to low and mid-rise buildings, analysis was performed and conclusions on the behavior of the high-rise buildings were drawn. The structural response was assessed to determine the overall safety of the structures under seismic demands. For better representation of the general seismic conditions, review of the past seismic activity in the nearby Balkan region and design legislation in Albania is also presented.

2 Past seismic activity in the nearby Balkan region

From geodynamic aspect, the Balkan Peninsula is one of the most active regions in Europe. There is evidence of intense seismic activity along the proposed cross-border region. Earthquakes with magnitude higher than six (M > 6) have occurred throughout the history (1906, Ohrid, ML = 6.00; 1911, Ohrid, ML = 6.70; 1912, Ohrid, ML = 6.0;, 1920, Vlore, ML-6.0; 1931, Valandovo, ML = 6.00; 1931, Valandovo, ML = 6.70; 1942, Debar, ML = 6.00; Skopje, ML = 6.10; 1960; Korcha ML = 6.4; 1967, Debar, ML = 6.50).

The effects of the Skopje major earthquake in 1963 were manifested by casualties and injuries, destruction and severe damage to a large number of buildings and other public and social facilities, damage to the infrastructure, lifelines, urban equipment, Fig. 1. Out of the total building area including dwelling houses (some 1,630,609 m2), 80.7% was destroyed or heavily damaged and about 75.5% of the inhabitants were left homeless. Only 19.7% of the structures remained not or slightly damaged and were usable immediately after the earthquake in accordance with the damage and usability criteria. The brick masonry wall structures of the buildings suffered more than any other type of structures and accounted for the larger number of deaths. Mixed structural systems also suffered considerably.

Fig. 1 (left) The Officers' Hall building, following the earthquake. Perhaps the most beautiful building that Skopje lost in the wake of the earthquake, (right) "Sloboda" square in ruins

Although many of these buildings did not collapse, they were left completely shattered, beyond repair. Old adobe structures, particularly those with timber bracings, resisted the shock with some damage but behaved far better than brick masonry or mixed structures. Reinforced concrete frame structures suffered comparatively little damage and only two small structures of this type collapsed. Tall frame structures, up to 15 stories, performed far better due to the specific frequency content of the earthquake. They were constructed with more care and, in some cases, wind forces were considered in the design. Finally, prestressed structures were totally destroyed after the collapse of their supporting columns. (Kapsarov 1972).

During the Bitola earthquake in 1994, 9884 individual buildings were inspected, including 66 schools, 5 hospitals, 198 important facilities and 32 religious structures. From all inspected buildings, 7467 buildings were damaged out of which 301 were important facilities and 7116 were individual buildings. Structural damages were found in 3 291 individual buildings and 143 important facilities (Petrovski et al 1994).

Between 1900 and 2014 in Turkey, 96,064 people lost their lives and 778,759 buildings were damaged or destroyed during 180 earthquakes causing fatalities and damage. The earthquakes causing fatalities and property loss during this period in Turkey occurred once in every four years. Most of the lives were lost during the earthquakes with magnitudes ranging between 7.5 and 7.9, while most of the buildings were damaged during earthquakes with magnitudes in the range of 7.0–7.4 (Bikçe 2016). The first five regions which have the most fatalities and property loss are ranked from most to least as follows: Eastern Anatolia, Marmara, Black Sea, Aegean, Mediterranean Regions. The occurrence of earthquakes with fatalities and property loss will be inevitable in future as it was in the past unless radical precautions appropriate to the tectonic features of the region are taken, since 70% of the population in Turkey lives in first degree and second-degree seismic zones. Learning from the experiences in the past earthquakes, fatalities and damages can be mitigated (Arslan and Korkmaz 2007).

The most affected structures by the catastrophic Vrancea (Romania) earthquake of 04.03.1977 with magnitude 7.2 were masonry, non-seismically constructed structures. Out of these, 46% were classified as temporarily useless. However, reinforced concrete structures were not spared by the earthquake either (Abolmasov et al. 2011). In Bucharest, the most extensive damage and even failure were observed in the central city area where 32 older 8 to 12 storey buildings (built prior to 1940) collapsed. In some of these buildings, poor quality concrete, low percentage of reinforcement of columns and beams and flexible ground floor with heavy bearing and partition walls over the ground floor, were detected. There were also structures with poorly repaired damages caused by the previous earthquake in 1940. The new reinforced concrete structures with light infill suffered slight and tolerable damage, particularly to the nonstructural elements, due to their improved strength and deformability capacity. Structures built prior to 1920 and those built between 1920 and 1940 suffered considerably more extensive damage, unlike structures built after the 1940 earthquake. As to the number of storeys, the structures with 3 storeys suffered the most severe damage due to which 49% of these, needed repair and strengthening.

The Montenegro earthquake of April 15, 1979 damaged 40,000 houses and buildings and utilities covering a total gross area of 5,634,088 sq.m. Out of this building stock, 16,331 (41.5%) covering a gross area of 1,912,514 sqm. (33.9%) were constructed on rock site conditions, whereas 23,049 (58,5%) buildings covering a gross area of 3,721,574 sqm. (66.1%) were constructed on diluvial-alluvial site conditions. The highest vulnerability to the seismic environment was shown by the class of masonry structures. It was concluded that the observed vulnerability of buildings constructed in rock site conditions was fairly higher than that of buildings constructed in diluvial-alluvial site soil conditions. It was also found that the observed vulnerability of masonry structures was generally increased with the increase of the number of storeys, but it was merely independent of the number of storeys in the case of RC classes of buildings (Petrovski et al. 1982). Damages caused by the 1979 earthquake with magnitude7.0 in Montenegro are shown in Fig. 2.

During the L'Aquila earthquake, the performance of the buildings was not satisfactory. Masonry buildings which were built with rubble stone, brick and hollow clay tiles suffered the most. Reinforced concrete structures suffered only where poor concrete quality, non-ductile detailing and strong beams-weak columns were present (Kaplan et al. 2010).

Many structural weak points were highlighted by the damages inflicted by the earthquake, including large openings in load bearing walls, lack of special vertical confining elements, poor quality mortar, heavy cantilever balconies and unconfined gable walls. The major cause of partial or total collapse of buildings were the hollow clay tiles used as brick units, the poor mud mortar, the insufficient anchorage between mud and stone, the lack of vertical confining elements and the inadequate connection of the load-bearing walls. Thousands of existing structures designed and constructed in accordance with earlier or no seismic codes at all, are present in seismically prone areas worldwide. These buildings must be properly retrofitted as soon as possible in order to prevent future loss of lives (Kaplan et al. 2010).

2.1 Design legislation in republic of Albania

Albania has a long history of code-regulated seismic design. The first seismic regulations, accompanied by the first seismic zonation of Albania, were adopted in 1952. The seismic design requirements were increased in 1963, while in 1978, no significant improvements were obtained with KTP 2–78. In the 20 year period extending from 1976 to 1995, the seismic activity of Albania was dominated by micro earthquakes and small earthquakes (Muço 1995). The two most distinct areas of activity were the sequence on the Montenegro–Albania border (including the shock of April 15, 1979) and the Nikaj–Merturi swarm in northern Albania, which lasted from November 1985 to August 1986 (Muço 1991). The vulnerability of the structures in the area is also due to large deformations of the ground. This is mainly due to the local soil conditions which increase the seismic design force.

On the other hand, an intensity of VIII MSK was assigned to the western part of Albania according to the valid regulations in the country, as given in Fig. 3. The codes which are valid (KTP-N.2–89) define the seismic force considering both soil types and building

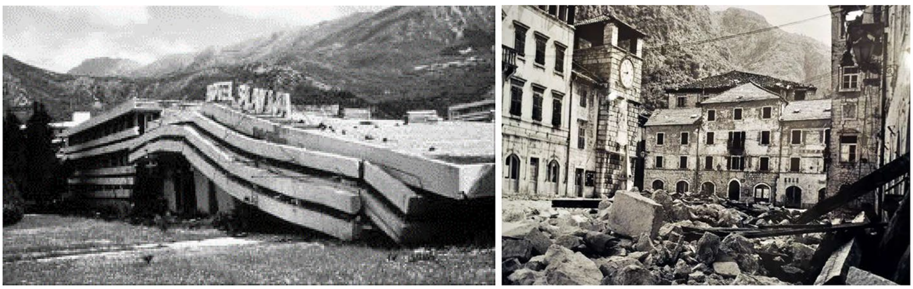

Fig. 2 Damage caused by the 1979 earthquake of magnitude 7.0 in Montenegro. (left) "Slavia" Hotel (Photo, by courtesy of Prof. Branislav Glavatovic, MSO, Montenegro.) (right) Old town of Kotor

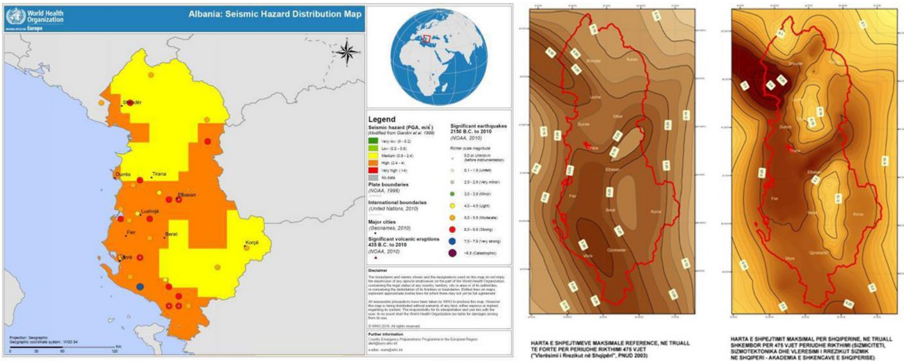

Fig. 3 Seismic Hazard Distribution Map (Earthquake Engineering Research Institute)

categories, although the construction practice often misinterprets the seismic force by reducing the coefficients related to importance, thus leading to flexible structures. The basis of the design concept is to provide a structure with a sufficient ductility, where the input energy will be dissipated through nonlinear cyclic deformations without compromising its integrity. According to the MSK-64 scale (Medvedev et al. 1965) (Fig. 4), the seismic hazard is defined through macro-seismic intensity areas, dividing the country into three large seismic zones with intensity VI, VII, VIII shown with different colors in Fig. 5. It also denotes some areas, located in the proximity of the epicenters of large seismic events,

Fig. 4 Geological map of Albania (1983) Scale 1:200.000 (Shehu et al., 1983)

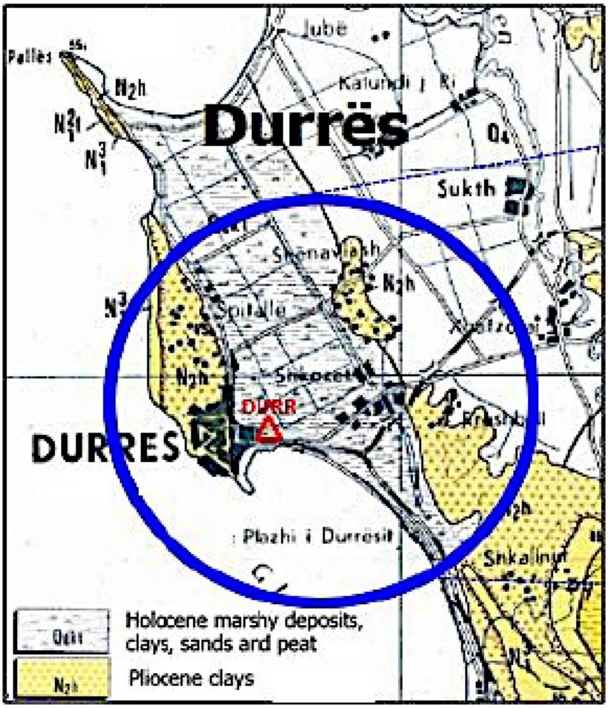

Fig. 5 Seismic zonation map of Albania (Sulstarova et al., 1980)

where the seismic intensity VIII is increased by one intensity level to IX at sites with poor soil conditions. For each of the identified seismic zones, three soil types are defined (i.e., I, II and III).

On 26.11.2019 at 02:54GMT, an earthquake with magnitude 6.4 hit western Albania, dramatically affecting the west part of the country, killing 51 people, injuring more than 900 and causing thousands of either total or partial damages to structures. The same earthquake was felt in the region. Based on the published fault planes, the earthquake was generated by the activation of the northwest-southeast striking fault. The aftershock sequence of earthquakes included more than 26 earthquakes with magnitudes around ML = 4.0 (Duni and Theodopoulidis 2019). The city that was the most affected was Durres, whose greater part lies on deposits of clays and sands. As can be seen from Fig. 4, the city of Durres is basically founded on Holocene deposits and Pliocene clays characterized by high amplification effects in agreement with the severity of damage observed in these areas.

The interesting fact is that the maps developed in 2003 (Fig. 3) are not required to be used in the design. PGA in the most active regions is between 0.24 g to 0.4 g, where 0.34 g is for Durres, categorized as the second highest risk in the country considering peak ground acceleration, urban stock size and population.

It can be stated that KTP-N.2–89 shares the common principles with modern seismic design codes such as Eurocode 8 (European Committee for Standardization (CEN) 2005), but it lacks several detailing recommendations. Generally, the structural response under the seismic design situation defined on the basis of EC-8 consists of higher force and displacement values compared to the respective values based on the Earthquake Resistant Design Regulations KTP-N.2–89, (Albanian Seismic Code). Despite the structural assessment of buildings built prior to the adoption of the Earthquake Resistant Design Regulations KTP-N.2–89, (Albanian Seismic Code), structural examination and reassessment of all buildings according to Eurocode-8 rules must be developed (Kokona et al., 2016). To overcome these issues, in the more recent years, the community of structural engineers has often used the good practices from Eurocode 8, integrating them into the KTP-N.2–89 in their every-day practice.

3 Verification analysis of damages to typical structural systems in Albania

To ensure a better understanding of the reasons for the structural and nonstructural damages that occurred during the recent earthquake, some structures must be further investigated and studied. After an overview of relevant literature, it can be seen that a very limited number of papers currently deal with analytical proof of the structural and nonstructural damages to residential buildings. The focus of this paper is on investigation of the reasons for structural failures by using an analytical approach based on the previously detected damages caused by the earthquake.

3.1 Description of building structure samples

To verify damages to structures that occurred during the Durres earthquake, typical reinforced concrete structures were analyzed as follows:

- Low-rise structures, Gf + 3 storeys with four different structural systems:
    - Low-rise regular frame structure (LRR-FS), (Fig. 6a);
    - Low-rise regular frame structure with shear-wall (LRR-FS-SW), (Fig. 6b);
    - Low-rise irregular frame structure with shear-wall (LRI-FS), (Fig. 7a);
    - Low-rise irregular frame structure with shear walls (LRI-FS-SW) (Fig. 7b).
- Mid-rise structures, Gf + storeys with two different structural systems:

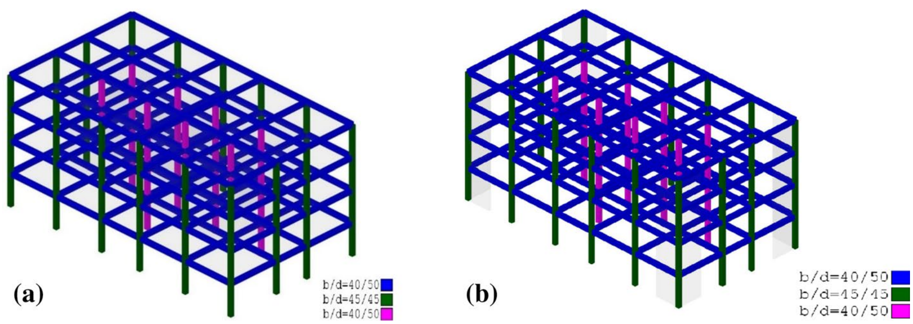

Fig. 6 Low-rise building models with dimensions of frame elements [cm]. **a** Low-rise regular frame structure (LRR-FS); **b** low-rise regular frame structure with shear-wall (LRR-FS-SW)

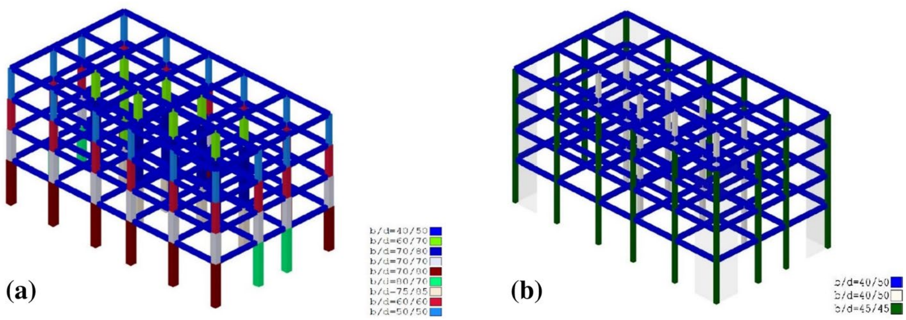

Fig. 7 Low-rise building models with dimensions of frame elements [cm]. **a** Low-rise irregular frame structure with shear-wall (LRI-FS); **b** low-rise irregular frame structure with shear walls (LRI-FS-SW)

  - Mid-rise regular frame structure (MRR-FS) (Fig. 8a);
  - Mid-rise regular frame structure with shear walls (MRR-FS-SW), (Fig. 8a);
- High-rise structure, Gf + 15 storeys
  - High-rise regular frame structure with shear walls (HRR-FS-SW), (Fig. 9).

All structures have the same regular plan, six longitudinal spans 6,0 m; 6,0 m; 5,0 m; 6,0 m; 6,0 m; and three transverse spans: 6,0 m; 4,0 m; 6,0 m.

3.1.1 Group 1–low-rise buildings

The first group represents structures with 4 floors or under. For this investigation, a structure consisting of a ground floor and three storeys was analyzed. This type of structures

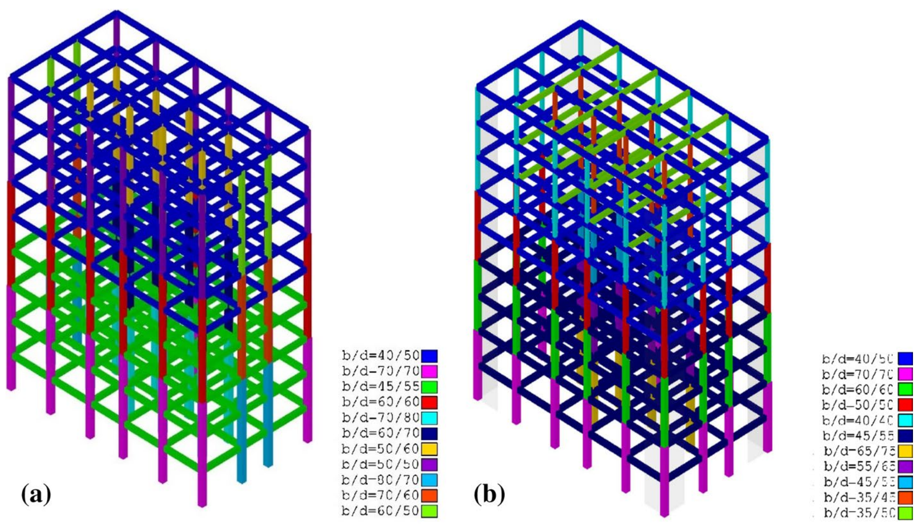

Fig. 8 Mid-rise building models with dimensions of frame elements [cm]. **a** Mid-rise regular frame structure (MRR-FS); **b** mid-rise regular frame structure with shear walls (MRR-FS-SW)

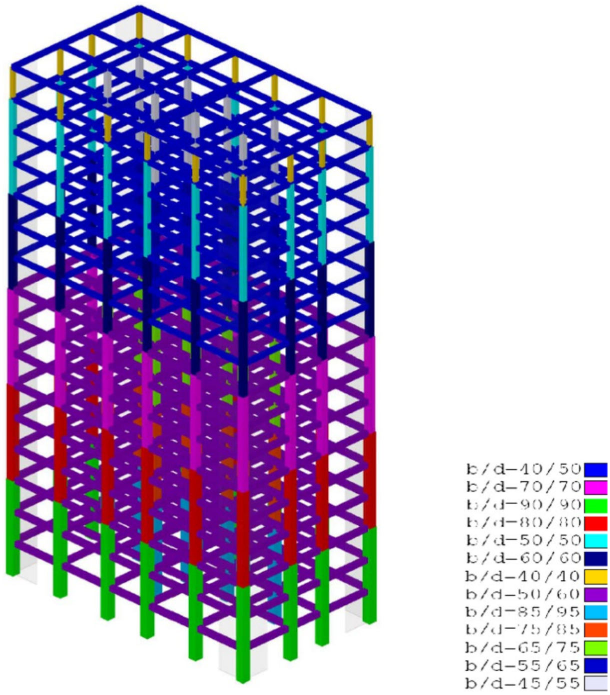

Fig. 9 High-rise building models with dimensions of frame elements [cm]

that are mostly used in the affected area experienced severe structural damage. In the first group, four different structures were analyzed. These four structures have the same regular plan, as all selected structures, the difference being in: the structural system (frame or frame-wall structural system) and the value of the selected ground floor height (two of these structures have a greater ground floor height that represents a soft storey). As to the frame-wall systems, the shear walls are placed in the corners of the structure, having a thickness of 20 cm and 3 m length, in both directions. The slab of all structures is 16 cm thick.

Figures 6 and 7 show the selected dimensions of elements (columns and beams) of each structure.

3.1.2 3.1.2. Group 2–mid-rise buildings

The second group of mid-rise structures is represented by nine storey buildings. This type of structures representing residential and/or commercial buildings are also commonly built in the affected area. For the second group, two structures of different structural system were analyzed. The two structures have the same regular plan as all selected structures, the difference being in: the structural system (frame or frame-wall structural system). In both structures, all storey heights are the same. In the frame-wall systems, the shear walls are placed in the corners of the structure, having a thickness of 20 cm and length of 3 m, in both directions. In all structures, the slab thickness is 16 cm.

Figure 8 shows the selected dimensions of the elements (columns and beams) of each structure.

3.1.3 3.1.3. Group 3–high-rise buildings

The third group–high-rise buildings is represented by sixteen storey structures. This type of structures was analyzed for the purpose of this paper. It is not commonly used in the affected area. One structure with a frame-shear wall structural system was analyzed. The structure has the same regular plan as all the selected structures, while all of its storey heights are the same. The shear walls are placed in the corners of the structure, having a thickness of 20 cm and length of 3 m, in both directions. In all structures, the slab thickness is 16 cm. Figure 9 shows the selected dimensions of the structural elements (columns and beams).

3.2 3.2 Applied methodology for structural analysis

For each of these models, the following steps of analysis were performed (Fig. 10):

1. Elastic, i.e., static and equivalent seismic force analysis by use of the fnite element method and SAP 2000 computer program (Tower Radimpex) in accordance with the national regulations (PIOVSP 1981). Infill walls were considered as permanent load, floor structures were modelled as SHELL elements and RC beams and columns were modeled as FRAME elements.

In the elastic structural analysis, the following material characteristics of the existing masonry were adopted:

- Specifc weight of concrete $\gamma = 25$ kN/m3
- Modulus of elasticity of concrete E = 3,300,000 kPa.

2. Analysis of elements up to the ultimate state of strength. Load and deformation capacity resulting in force–displacement (Q–δ) relationships for each element and cumulatively for each story, displacement capacity δu and ductility capacity defined as $\mu = \delta u/\delta y$, were considered. The displacement and ductility capacities were defined using the methodology and a corresponding package of in-house computer programme for optimal design of new and performance evaluation of existing structures. Structural systems of the selected buildings were analyzed up to ultimate state. Strength and deformation capacity of each RC element (including RC shear walls) expressed through the

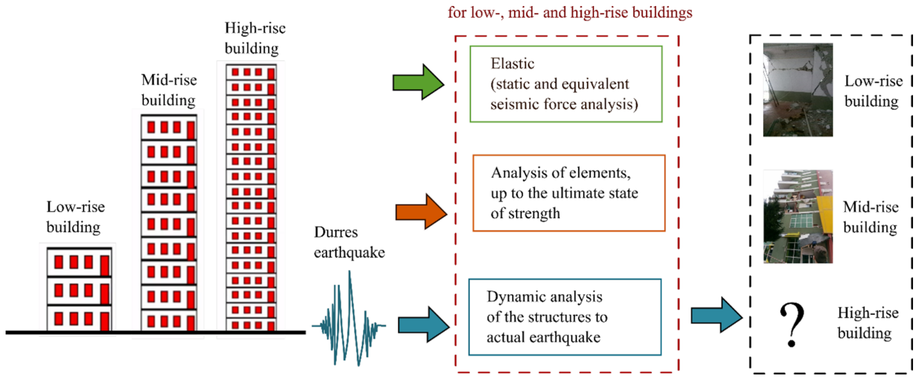

Fig. 10 Applied methodology scheme

capacity curve (Q–δ relationship) was calculated based on the theoretical background of strength of members with flexure and axial load and ultimate deformation and ductility of members with flexure (Park and Paulay 1975). In order to calculate the real strength and deformability capacity of each element, depending on the quality and quantity of built-in materials (concrete and reinforcement), an in-house computer programme was developed at IZIIS (Necevska-Cvetanovska and Petrusevska 2000; Necevska-Cvetanovska and Apostolska 2012). First of all, the moment-rotation relationship at the yielding and the ultimate point was developed and then, for each element of a given storey, the displacements and the shear forces at the yielding (δy and Qy) and the ultimate point (δu and Qu) were calculated.

3. Analysis of the dynamic response of the system under an actual earthquake. The intensity and frequency content at the considered location were defined by applying the in-house developed software package, (Necevska-Cvetanovska and Petrusevska 2000; Necevska-Cvetanovska and Apostolska 2012). The verification of the IZIIS methodology and the in-house developed programme package for design of new and assessment of existing building structures was performed using the computer programme IDARC2D (Park et al. 1987; Necevska-Cvetanovska and Cvetanovska 2010). The main dynamic model of the RC building structure represents a fixed-base schematized system of "n" masses concentrated at story levels that have connections that allow displacement only in horizontal direction ("shear-type" model). The application of such simplified modeling is fully justified for structures with rigid diaphragms, which enable equal horizontal displacements at story levels due to column bending (rotations of joints are constrained). The nonlinear variation of stiffness (force–displacement relationship) at the story, which is considered as a "macro element" in the course of time is defined through the corresponding hysteretic models. The story force–displacement relationship was obtained by summing up the force–displacement relationships of all RC vertical elements (columns and shear walls) for the RC structures, or all masonry walls (for the masonry structures), as explained above, at the corresponding level. Definition of the system response for such an idealized structure and for random ground displacement was reduced to solving a system of differential equations of the second order with variable coefficients in the course of time. The input data for the mathematical model of the system, at story level, were: masses, storey stiffness, displacement and shear forces at yielding and ultimate point, and the earthquake record in the form of time history of ground acceleration. The output for the analysis included: time histories of acceleration, velocity and relative displacements, at each story level, as well as the maximum values of these parameters.

Determination of ductility capacity and required ductility By summing up the Q–δ relationship of all elements at a given story (as explained above), the Q–δ relationship at that level was obtained. The deformability capacity of the story was defined via the displacement at the ultimate point, δu. The ratio between that displacement and the displacement at the yielding point represents the ductility capacity of the story, $\mu = \delta u/\delta y$. The response of the structure (observed at the same story) expressed via displacement due to the imposed ground motion δRQ (demand) compared with the displacement capacity, defines the seismic performance of the structure. The δ_{RQ}/δ_y ratio represents the so called required ductility.

According to this:

- If $\delta_{RQ} \le \delta_y$, i.e. $\delta_{RQ} \le 1$—the structure is in the elastic range;
- If $\delta y \ge \delta_{RQ} \le \delta_u$ i.e. $\mu \ge \mu_{RQ} > 1$—the structure is in the nonlinear range;
- If $\delta_{RQ} > \delta u$, i.e. $\mu_{RQ} > \mu$—the structure experiences failure.

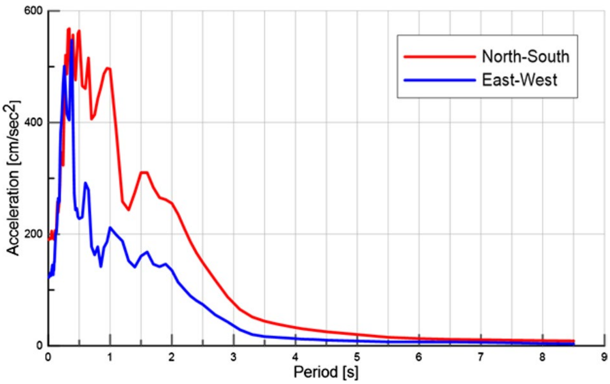

Fig. 11 Elastic response spectra of the main shock in Durres composed of N-S and E-W components according to the Durres station record

Table 1 Fundamental periods of vibration of low-rise structures

	Fundamental period of vibration (sec)			
	LRR-FS	LRR-FR-SW	LRI-FS	LRI-FS-SW
T_1	0.332	0.243	0.486	0.457
T_2	0.316	0.237	0.461	0.447
T_3	0.294	0.207	0.421	0.432

The dynamic analysis was performed by using the real recorded acceleration in Durres (station DURR) located 15.6 km from the epicentre (Freddi et al. 2021). The station is located on ground type C according to Eurocode 8, with average shear wave velocities of $V_{s30} = 202$ m/s in the upper 30 m. Figure 11 shows the elastic response spectra of the main shock in Durres, with 5% of damping coefficient used in the nonlinear dynamic analysis of the typical structural systems.

3.3 Results from analyses

For the selected structures, the obtained displacement/ductility capacities compared to the demanded ones for the corresponding earthquake with intensity and frequency content are illustrated herein. The blue colour represents the achieved ductility per stories, while the red color shows the ductility capacity of the structure per stories. The green colour indicates the displacement per stories caused by the earthquake excitation, while the yellow colour points to the displacement capacity of the structure per stories.

3.3.1 Group 1

From the first group of structures, a low-rise regular frame structure (LRR-FS), a low-rise regular frame structure with shear-walls (LRR-FR-SW), a low-rise irregular frame structure (LRI-FS) and a low-rise irregular frame structure with shear walls (LRI-FS-SW) were analyzed.

The fundamental periods of all analyzed structures belonging to group 1 are shown in Table 1.

3.3.1.1 Low-rise regular frame structure The nonlinear dynamic analysis was based on input data from the equivalent seismic force analysis. Figure 12 shows the relative story displacements and ductility demand in x-x and y–y direction for the design earthquake ($a_{max} = 0.21$ g). The ductility demands were defned by use of the methodology and a corresponding package of computer programs for performance evaluation of existing structures, (as mentioned in item 3.1). The results showed that, in the case of this type of structures, the capacity at the first and the second level was exceeded in both x and y directions. A particular difference was observed at the first level where the ductility capacity was exceeded almost thrice, while the displacement capacity was exceeded more than thrice. Due to these differences, heavy damages to nonstructural elements with possible occurrence of damages to structural elements in the lower storeys as well as possible disturbance of the global stability of the structures were expected to occur (Fig. 13).

3.3.1.2 Low-rise regular frame structure with shear-walls Unlike frame structures with a height regularity, in this type of structures, the displacement and ductility capacity were exceeded in both x and y direction, the difference in respect to purely frame structures being that the exceedance was approximately twice. Uniform exceedance at both the ground floor and the first storey was observed particularly in respect to displacements, which was not the case with the regular structures without shear walls. Due to the presence of shear walls, the great difference in displacement capacity was reduced. Such performance of this type of structures may cause damage to nonstructural elements and possible slight damage to structural elements by which the stability of the entire structural system is not endangered.

3.3.1.3 Low-rise irregular frame structure In the case of this type of structures, the earthquake that occurred in Durres caused displacements that were smaller than the capacity of the structure (Fig. 14) whereat the earthquake did not cause exceedance of the ductility of the structure. Therefore, neither nonstructural nor structural damage was observed in this type of structures. Under the effect of the occurred earthquake, these structures remain in the linear range of behaviour and generally exhibit the most favourable behaviour. Such behaviour is due to the specific characteristics of the structures of this type, enabling their favourable behaviour during an earthquake with the characteristics of the considered earthquake.

3.3.1.4 Low-rise irregular frame structure with shear walls This type of irregular structures show an extraordinary exceedance of displacements and capacity at the first storey in both directions (Fig. 15). At the second storey, the capacity of both displacement and ductility is almost identical with the displacements and ductility caused by the occurred earthquake. In this type of structural system, nonstructural damage is observed particularly at the first level.

3.3.1.5 Summary of results on low-rise buildings Almost all analyzed structures with different structural system show potential damage to the first level, with deformations and ductilities that are considerably higher than the capacity of the structures and the acceptable ones by the regulations. During an earthquake, such deformations cause heavy structural damages up to unusable structures, or necessity for their tearing down or repair and strengthening at huge costs. The knowledge obtained from these analyses of the performance of such structures completely corresponds with the damages found during the visual inspection of modern RC buildings in Durres.

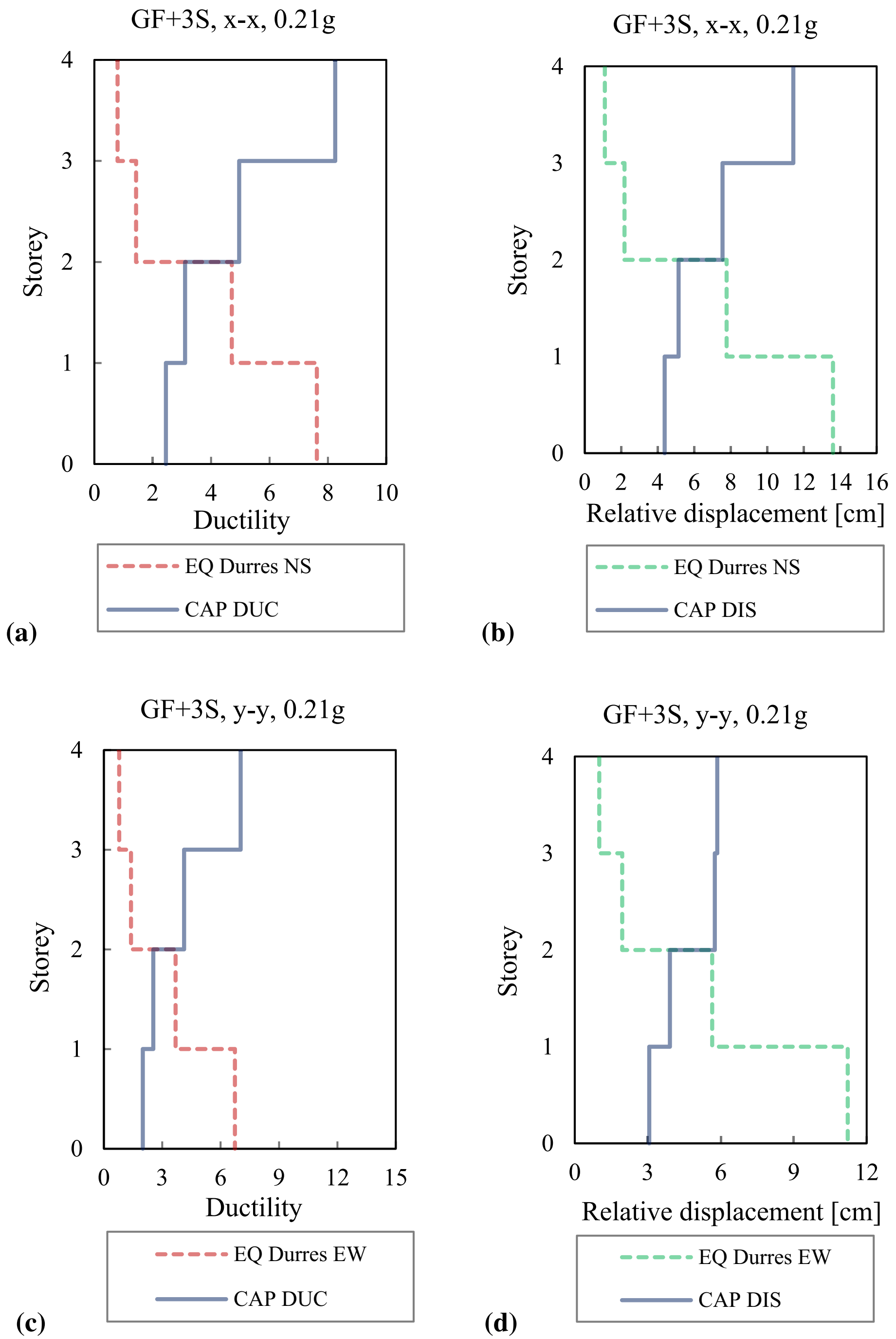

Fig. 12 Graphical presentation of the results for LRR-FS. **a** Storey–ductility in x direction; **b** storey–displacement in x-direction; **c** storey–ductility in y-direction; **d** storey–displacement in y-direction. Blue–achieved ductility per storeys under Durres 2019 earthquake, red–ductility capacity of the structure per storeys, green–displacement per storeys caused by earthquake excitation, yellow–displacement capacity of the structure presented per storeys

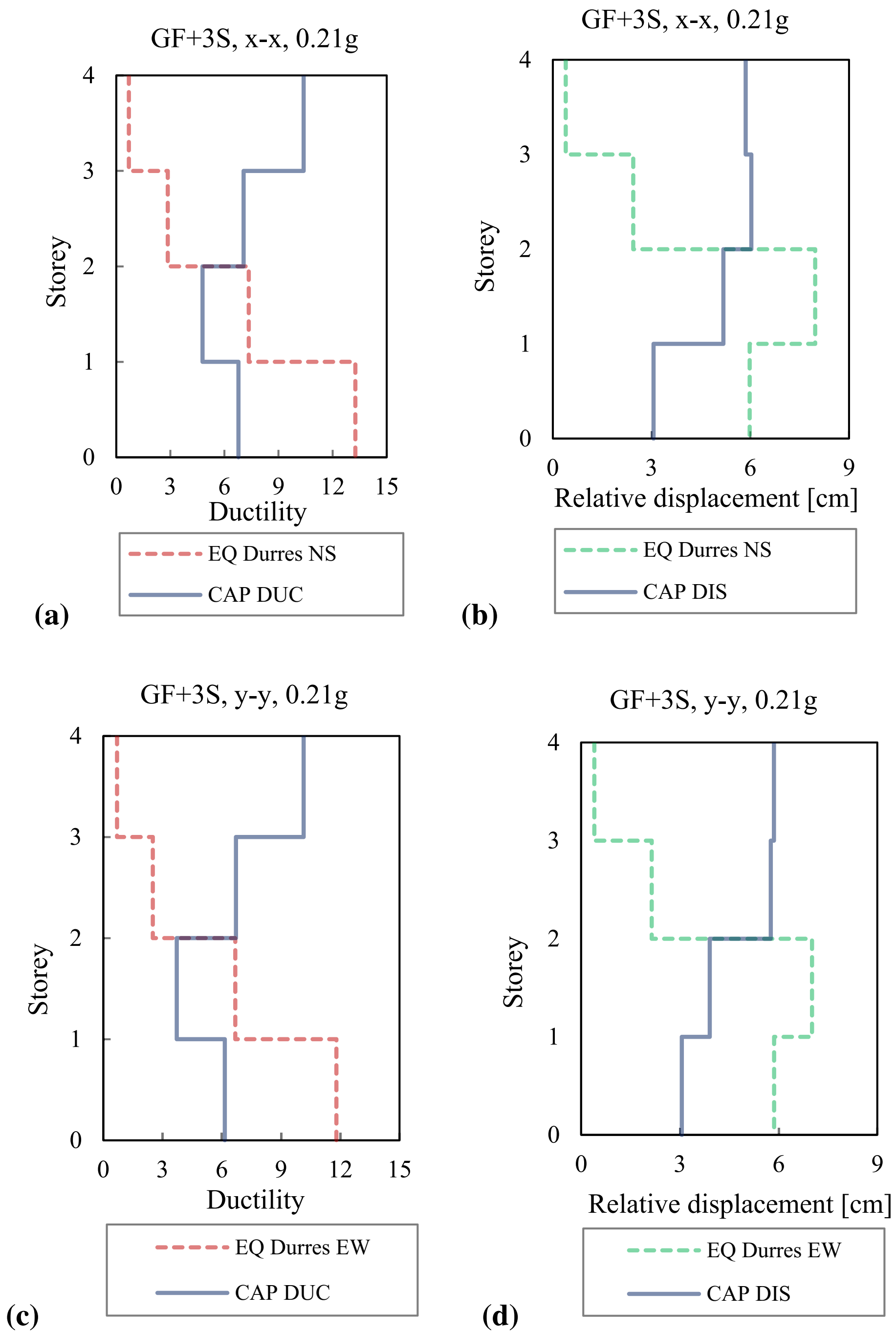

Fig. 13 Graphical presentation of the results for LRR-FR-SW. **a** Storey–ductility in x direction; **b** storey–displacement in x-direction; **c** storey–ductility in y-direction; **d** storey–displacement in y-direction

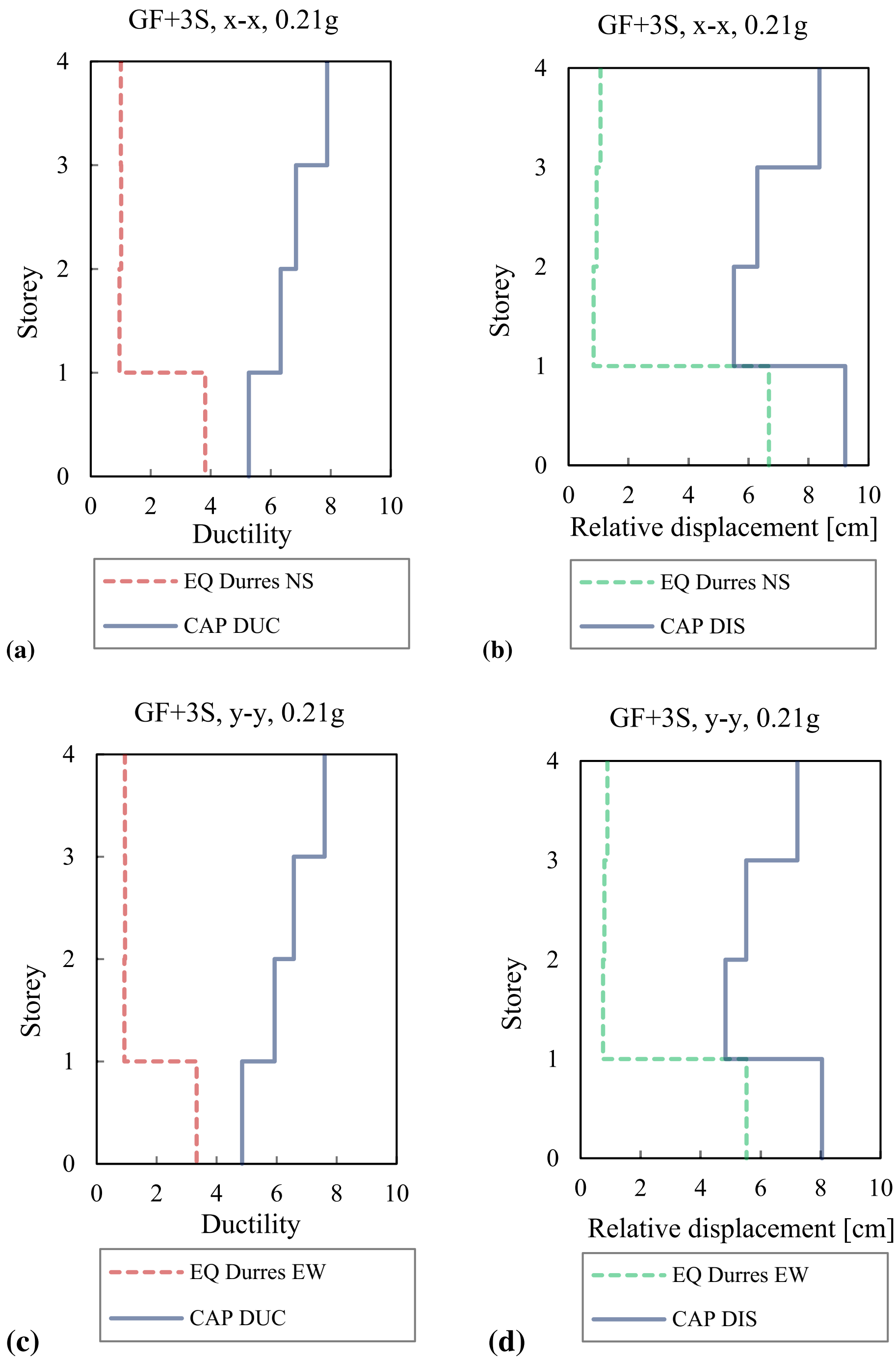

Fig. 14 Graphical presentation of the results for LRI-FS. **a** Storey–ductility in x direction; **b** storey–displacement in x-direction; **c** storey–ductility in y-direction; **d** storey–displacement in y-direction

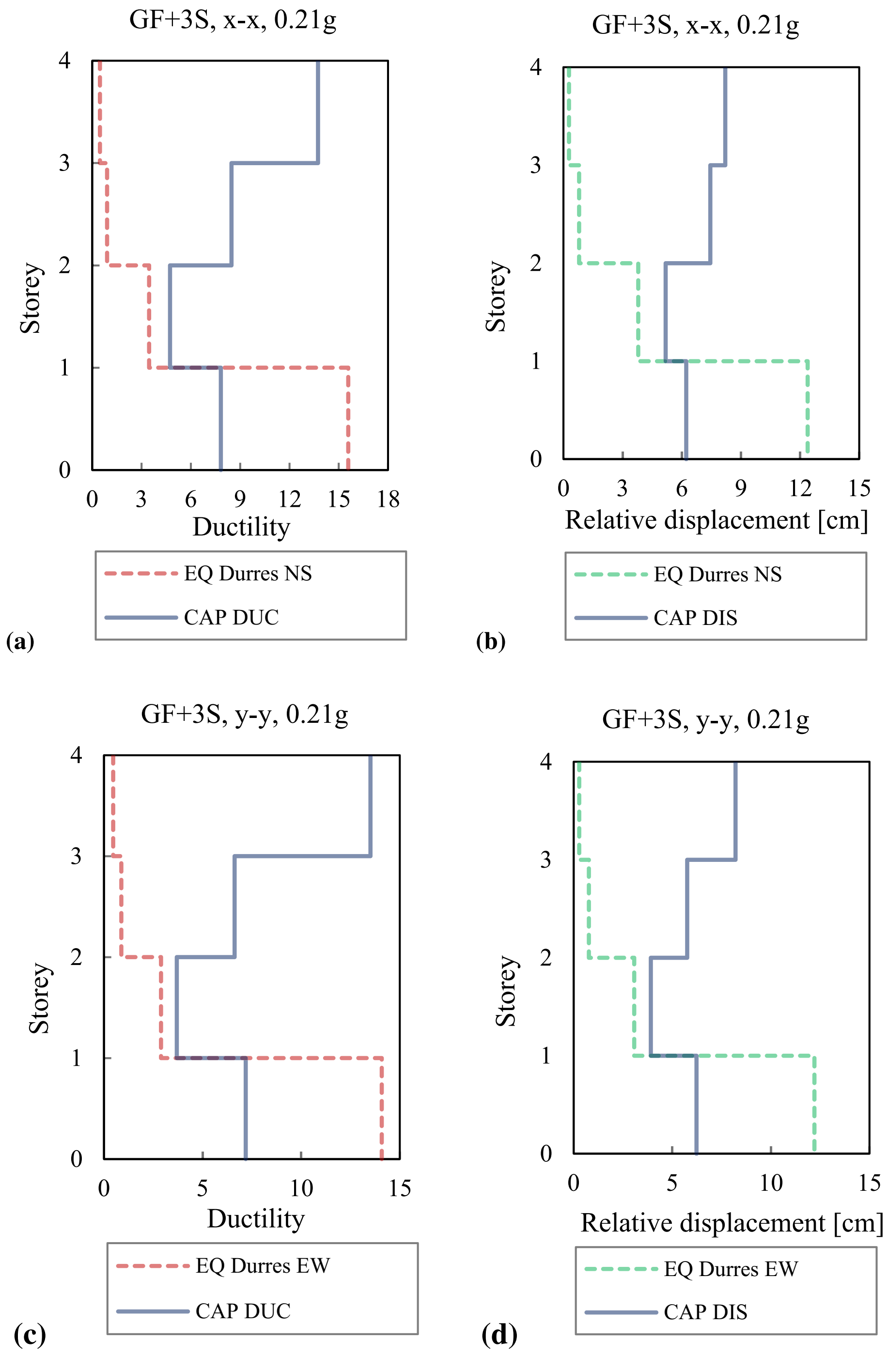

Fig. 15 Graphical presentation of the results for LRI-FS-SW. **a** Storey–ductility in x direction; **b** storey–displacement in x-direction; **c** storey–ductility in y-direction; **d** storey–displacement in y-direction

Fig. 16 Low-rise buildings with heavy structural damage to the first level after the Durres earthquake

Fig. 17 Damaged columns at the ground floor of a low-rise building after the Durres earthquake

The reason for this is that most of the low-rise structures whose fundamental period of vibration is relatively close to the period of vibration of the earthquake are logically expected to experience a state of resonance. This is confirmed by the results obtained for a total of 3 out of 4 such structures. The knowledge on the behavior of the analyzed structures fits into the manifested damage to modern RC structures of this type in Durres.

These structures are usually designed by mediocre design engineers, while their construction most frequently lacks sufficient engineering control. This was observed in the case of the 3-storey structure for commercial/residential use shown in Fig. 16.

The structure manifested heavy structural damage only due to inappropriate construction, or more precisely, due to low quality of concrete and embedded stirrups with an improper diameter as well as improper overlapping distance and length (Fig. 17). This contributed to a regular frame structure to be built so as to suffer such damage. In this case, a question is posed as to the necessity for optimization of the structural systems in designing for an earthquake record defined for a corresponding location.

3.3.2 Group 2

The fundamental periods of all analyzed structures of Group 1–low rise building structures are shown in Table 2.

3.3.2.1 Mid-rise regular frame structure From the performed analyses of mid-rise regular structures and the results obtained (Fig. 18), it can be concluded that the displacement and ductility capacity is not exceeded up to the 4th level, up to which the displacement and ductility capacities have an approximate value. The greatest exceedance takes place at the topmost storeys.

3.3.2.2 Mid-rise regular frame structure with shear walls The results from the analyses of mid-rise regular frame structures with shear walls presented in Fig. 19 show that exceedance of the ductility and displacement capacity starts from the third level and it is exceeded up to the seventh level. At both topmost storeys, there is no exceedance of bearing and displacement capacity.

3.3.2.3 Summary of results on mid-rise buildings The knowledge gathered from the performed analyses of the behaviour of all mid-rise buildings points to acceptable dynamic behaviour with minimal damage to the ground floor, with deformations and ductilities smaller than the capacity of the structures mainly at the first storey heights, within the frames of the displacements and ductilities allowed by the regulations. During an earthquake, such deformations do not cause heavy structural damage to the most loaded parts of the structures. However, the behaviour of higher storeys, from the fourth to the seventh storey, shows relatively large deformations and ductilities that are larger than the capacities of the structures and also larger than those acceptable by most of the regulations worldwide and in Europe. Although the structures will not experience failure, they are still not safe for living due to large deformations, the inability for their use due to extensive damage to nonstructural elements–façade and partition walls as well as damage to the installations. The repair and strengthening of these structures and their retrofitting into stable conditions requires a specific approach to the technical solution, specialized labour force, materials and control, which on the other hand, is very much time consuming and costly. In the meantime, the structures cannot be used.

These results completely correspond to the manifested damages to modern mid-rise RC structures in Durres (Fig. 20), the difference being that these suffered considerably larger deformations and damages due to inconsistent application of recent knowledge in design, construction and quality control of earthquake resistant structures. Structural flaws in design and construction as well as inappropriate quality of built-in materials were also observed (Sheshov et al. 2021).

Table 2 Fundamental periods of vibration of mid-rise structures

	Fundamental period of vibration (sec)	
	MRR-FS	MRR-FS-SW
T_1	1.144	1.024
T_2	1.003	0.916
T_3	0.802	0.776

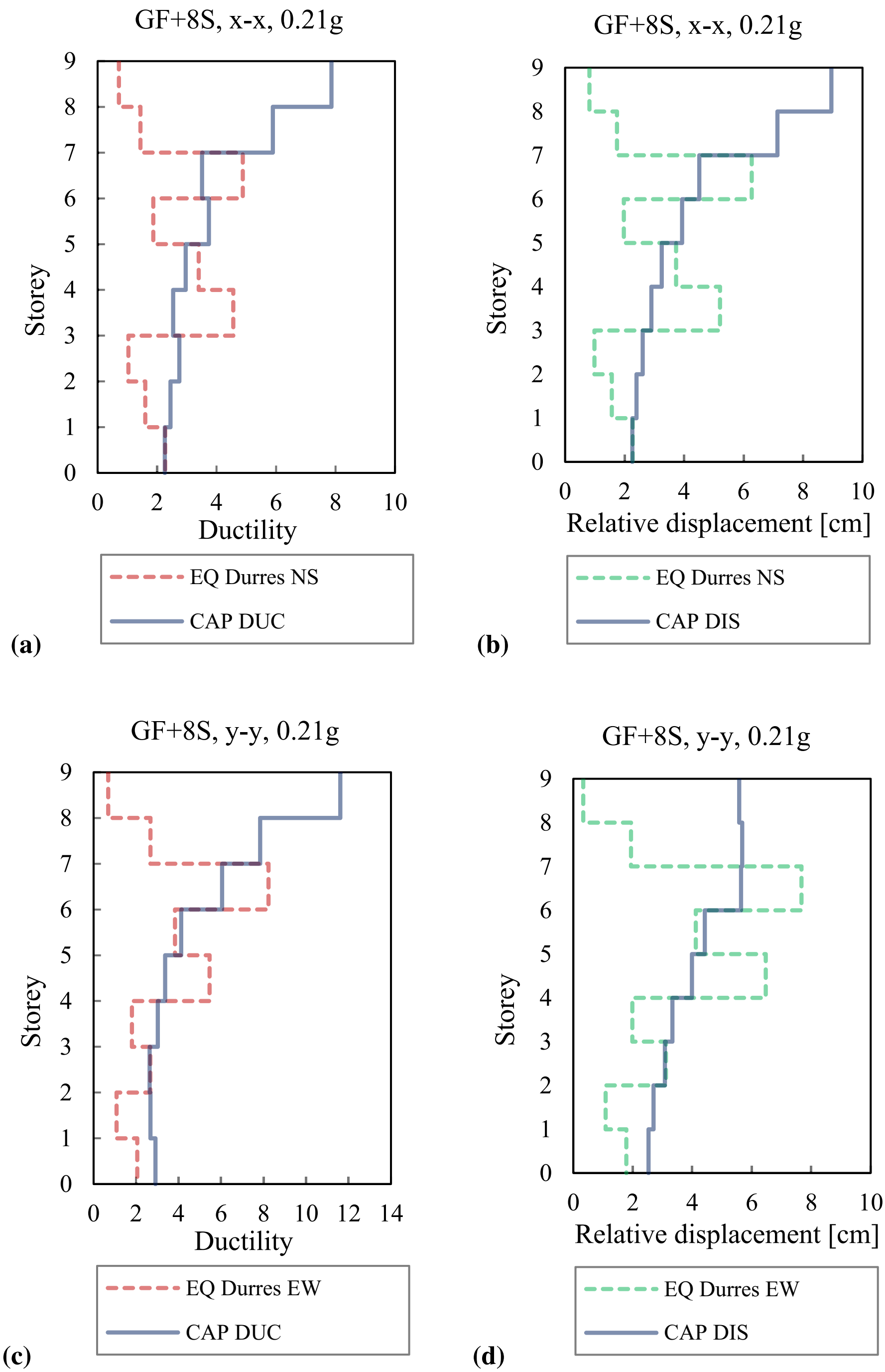

Fig. 18 Graphical presentation of the results for MRR-FS. **a** Storey–ductility in x direction; **b** storey–displacement in x-direction; **c** storey–ductility in y-direction; **d** storey–displacement in y-direction

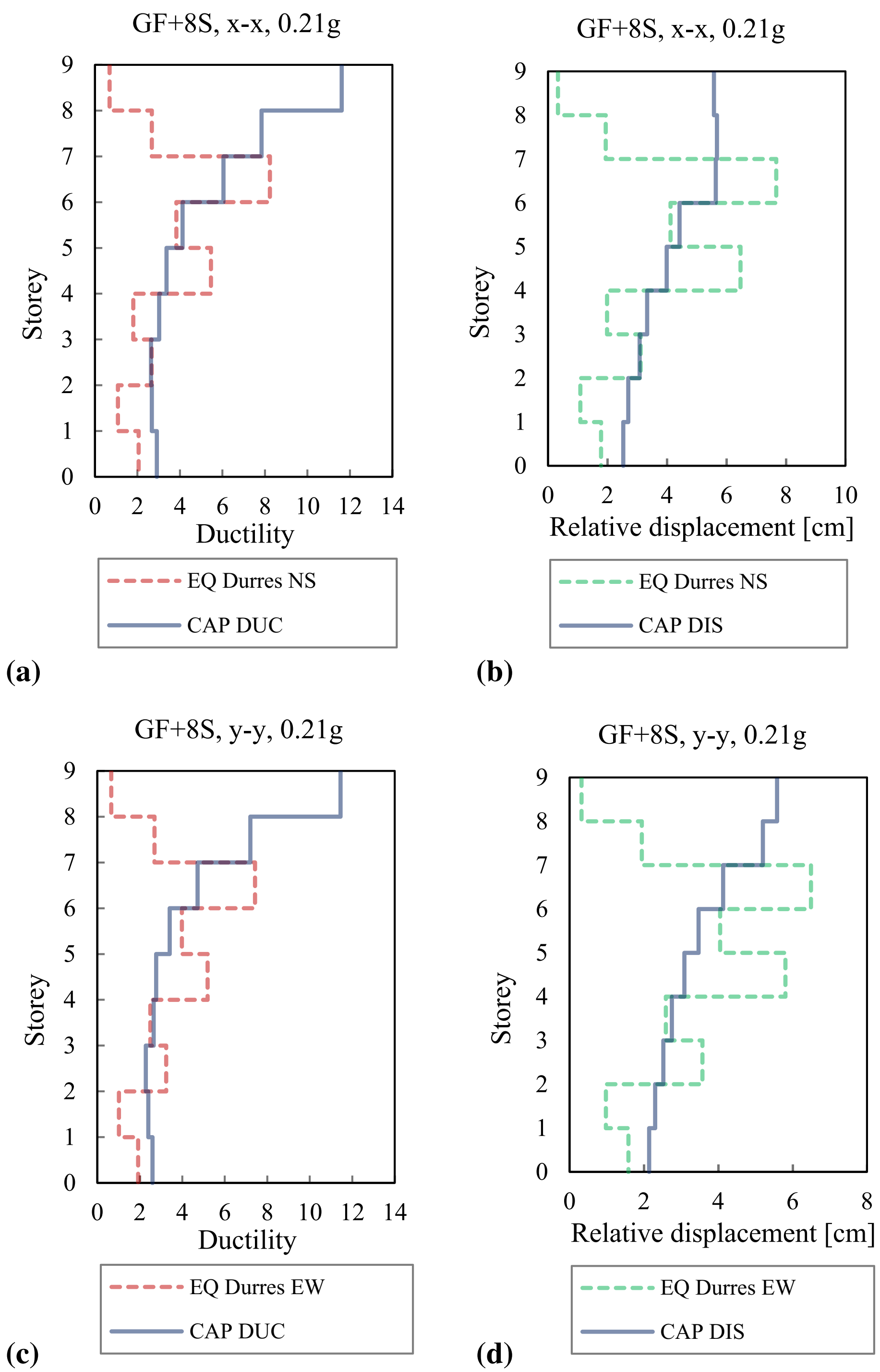

Fig. 19 Graphical presentation of the results for MRR-FS-SW. **a** Storey–ductility in x direction; **b** storey–displacement in x-direction; **c** storey–ductility in y-direction; **d** storey–displacement in y-direction

Fig. 20 Mid-rise building damaged after Durres in Albania, earthquake 2019. **a** and **b** (Sheshov et al. 2021); **c** (https://www.caritas.eu/earthquakes-in-albania)

The repair and strengthening of these structures and their retrofitting requires a specific approach to the technical solution, a highly qualified labor force, materials and control, which on the other hand, is very much time consuming and costs too much. In the meantime, these structures cannot be used.

The knowledge gained from the performed analysis of the behavior of the analyzed structures fits into the manifested damage to the modern RC structures in Durres, the difference being that these structures suffered considerably larger deformations and damages due to the use of flat-slab systems (Fig. 20).

3.3.3 Group 3

Table 3 shows the fundamental period of vibration of the analyzed high-rise structures.

3.3.3.1 High-rise regular frame structure with shear walls This group of structures was analyzed for the purposes of this paper although such type of structures were not observed in the region struck by the earthquake in Durres in 2019. According to the results obtained from the performed analyses, it can be concluded that the behaviour of these structures is not much different than that of the mid-rise buildings (Fig. 21) Similarly to these structures, greater deviations in displacements and ductility take place in the middle storeys, while the displacements and the ductility of the topmost storeys are below the capacity of the structure.

Table 3 Fundamental periods of vibration of high-rise structures

	Fundamental period of vibration (sec)	
	MRR-FS	MRR-FS-SW
T_1	1.144	1.024
T_2	1.003	0.916
T_3	0.802	0.776

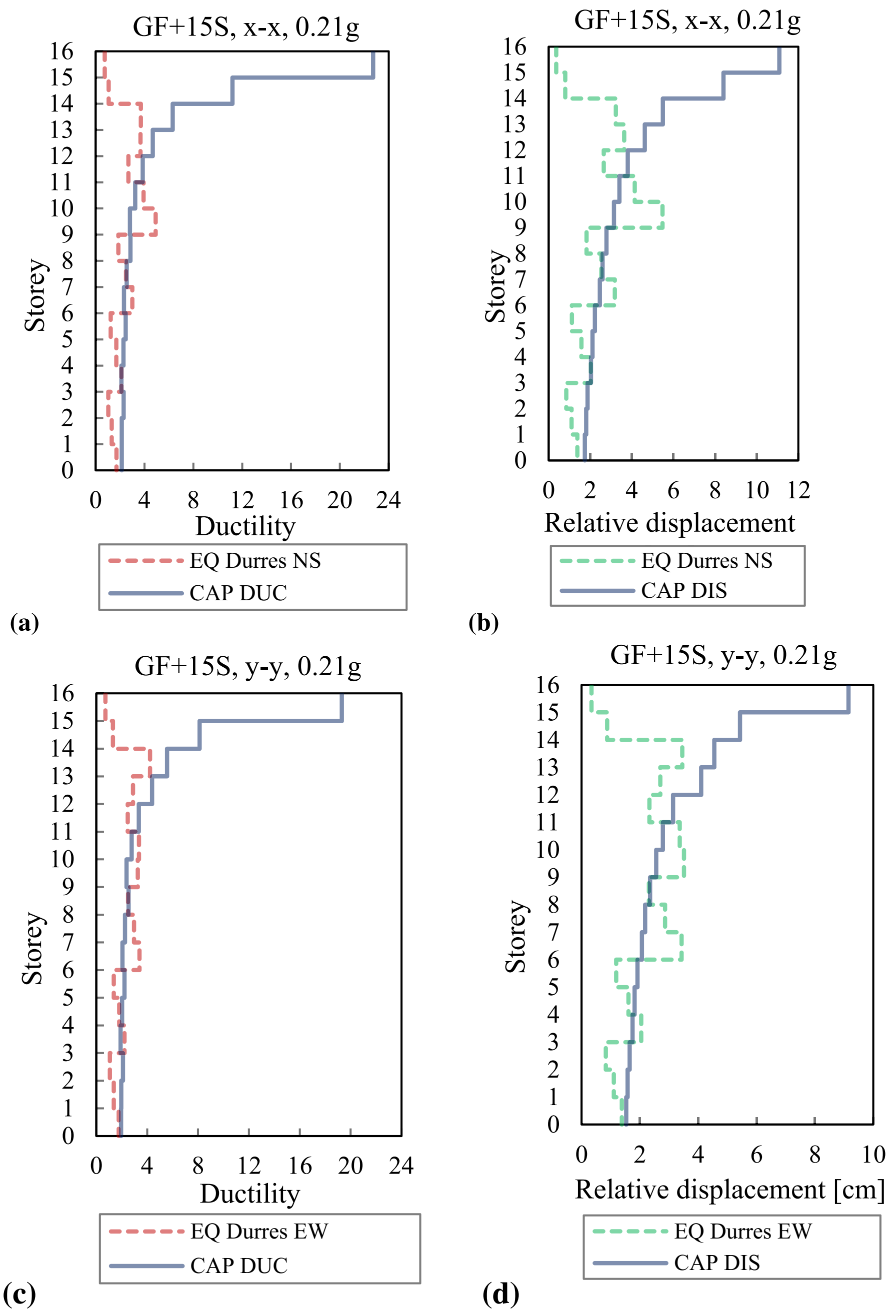

Fig. 21 Graphical presentation of the results for the 16 storey structure with height regularity and frame–shear wall structural system: **a** storey–ductility in X-X direction; **b** storey–displacement in X-X direction; **c** storey–ductility in Y-Y direction; **d** storey–displacement in Y-Y direction

4 Summary of the analysis

To apply the methodology for design of stable and low-cost reinforced concrete structures with controlled and conducted ductile behavior, it is essential to go one step further than the requirements in the technical regulations defining the dynamic response of a structure to real seismic effects, as a criterion for assessing the stability and the degree of vulnerability. The study presented herein deals with verification of the behavior of structures exposed to the recorded ground motion in Durres. Considering that the Durres earthquake response spectra spread over a wide frequency range, structural and non-structural damages were expected and occurred in low-rise and mid-rise RC structures. Using the same methodology of analysis of structures for high-rise structures, their seismic performance was predicted. The summarized conclusions for each group of structures are given separately further in the text.

4.1 Group 1–low-rise buildings

- Four different structural systems, namely, regular and irregular frame structures and regular and irregular frame structure with shear-walls, were analyzed.
- The results from the performed analyses of regular frame structures show that, in both x and y direction, the capacity of ductility and displacement is exceeded at the ground floor and the first storey. Particular difference in displacements is observed at the ground floor. In x direction, the displacements at the ground floor caused by the earthquake are thrice larger, while in y direction, they are four times larger. At the first storey, the difference in displacements is smaller. These differences at the ground floor cause heavy damages to nonstructural elements and occurrence of damages to structural elements, with possible disturbance of the global stability of the structure that leads to its uselessness and necessity for its demolition or repair and strengthening at enormous costs.
- In the case of regular frame structures with shear-walls, the displacement and ductility capacity is exceeded, but unlike the preceding structures, in these structures, there is a greater exceedance of the ductility capacity compared to the exceedance of the displacement capacity. In this type of structures, there is not so drastic difference in exceedance of the capacity at the ground floor compared to the first storey. In both x and y direction, at the ground floor and the first storey, the exceedance of the displacement capacity is approximately the same and can cause damages to nonstructural elements and possible damages to structural elements, without disturbance of the global stability of the structure.
- According to the analyses performed, the most favourable behaviour during the earthquake is exhibited by the irregular frame structures. These behave in the linear range.
- Irregular frame structures with shear walls behave unfavourably, particularly at the ground floor. Above this level, the structure behaves in the linear range.

4.2 Group 2–mid-rise buildings

- In both structures without walls and structures with walls, the greatest damage is expected to occur at their middle storeys. In the case of the analyzed structure, the most

extensive damage to nonstructural elements is expected and was observed between the third and the sixth storey. Both nonstructural and structural damage is possible, but the stability of the structure is not endangered.
- The analyzed structures showed an acceptable dynamic behavior with minimal damage to the lower floors, with deformations and required ductility smaller than the capacity of the structures, mainly at the first storey. The storey displacement and the required ductility generally fulfill the regulations. During an earthquake, such deformations do not cause heavy structural damage to the most loaded parts of the structures. Still, the middle storeys suffer displacements beyond their capacities and those acceptable by most regulations in the region and worldwide.

4.3 Group 3–high-rise buildings

- This type of structures are expected to behave similar to mid-rise structures regarding their middle levels. The particular analyzed structure was expected to suffer damages above the sixth storey.
- The middle storeys suffered displacement and required ductility larger than the achieved. Given that there was no damage assessment for structures of such heights in Durres, there was no opportunity to conclude whether this phenomenon also takes place in structures with and beyond this height.

5 Conclusions

This paper presents an analytical proof of structural and nonstructural damage to low-rise and mid-rise buildings in the area of Durres and Shijak caused by the earthquake that took place in Durres (Albania) in 2019. The results from the performed analyses of different types of structural systems of different height are presented. The general conclusion drawn on the basis of the results obtained is that the earthquake in Durres causes unfavourable behaviour of all structures, regardless their height and structural system, particularly low-rise structures, as shown by the on-site inspection. Unlike low-rise structures, high structures exhibit a larger capacity for deformations wherefore no structural damage to these structures was observed.

The results showed that low-rise buildings, unlike other buildings, exhibited the greatest capacity disbalance wherefore they suffered structural and nonstructural damage. In regular low-rise structures, there are relative displacements that are much greater than the structural capacity, leading to extensive nonstructural and structural damage, particularly at the ground floor and the first storey. In the case of structures with a shear wall, the difference in displacements per all the storeys is smaller than that in the case of purely frame structures, but still exceeds the displacement capacity of the structure. The most favourable behaviour is exhibited by low-rise irregular frame structures. In these structures, the greater height of the first level enables a larger ductility and displacement capacity. The damages to low-rise buildings in the stricken region confirm the results obtained from the analyses, the damage to this type of structures being much more expressed. The reason for this is that the fundamental period of vibrations of the structure is relatively close to the period of vibrations of the earthquake so that a resonance state is expected.

The results obtained for the mid-rise buildings show that the most affected are the middle storeys. For the buildings having 9 storeys (Gf + 8), these include the third storey up to the seventh storey, regardless the type of the structural system.

The third group–high-rise buildings was analyzed for the purposes of the presented investigations, although this type of structures was not found in the stricken region. The results from the analyses of these structures show that they exhibit behaviour that is similar to that of the mid-rise buildings, i.e., the most extensive damage to these buildings is also expected to occur at the middle storeys.

From the performed investigation of the dynamic response of all types of structures, it can be concluded that, even in the case of relatively well conceptualized structural systems, the dynamic response of the structures is relatively unfavourable. If one also takes into account the additional inconsistencies as early as in the process of design and later in construction of the structures in Durres, then one can confirm, with a certainty, the reason for the incurred large number of damages to RC structures.

Funding The authors have not disclosed any funding.

Declarations

Conflict of interest The authors have not disclosed any competing interests.

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