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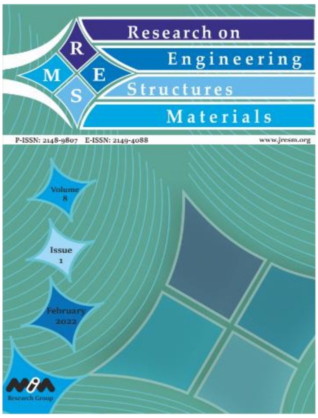

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Research Article

Damage potential of near and far-fault ground motions on seismic response of RC buildings designed according to old practices

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Abstract

This paper presents the evaluation of the seismic response of reinforced concrete (RC) residential buildings with the selected template designs in Albania considering their inelastic behavior of RC components. Four residential buildings having 5- and 6-story heights with template designs were chosen to represent the building practice in Albania before the adoption of today's modern seismic codes. Selection of the buildings and the material characteristics were based on site investigations after the November 26, 2019 earthquake sequences in several cities of the country. Pushover and dynamic analyses were deployed in both principal directions to obtain the seismic capacities of the selected buildings. The earthquake demands are evaluated comparatively under a set of far-fault and near-fault ground motions and the nonlinear dynamic characteristics were calculated using equally single degree of freedom (ESDOF) system approach. The impact of the material quality on the seismic response of the residential buildings were analyzed. Reasons of the observed building damages during the recent Albanian earthquakes were examined using the results of the performance evaluation of the selected buildings. The detailed analysis of the pushover curves and performance assessment identified the deficiencies and possible solutions for the studied typologies.

1. Introduction

Recent earthquakes in populated areas of the world have had a major impact on civilian structures designed and built according to pre-modern codes of practice, revealing that these buildings are seismically inadequate [1- 4]. Various devastating earthquakes, notably the 1989 and 1994 California earthquakes (Loma Prieta and Northridge), the 1995 Japan earthquake (Kobe); the 1999 Turkey (Marmara), 2009 and 2012 Italy (L'Aquila and Emilia Ramagna) and 2019 Albania earthquakes caused significant damage to the built environment. After these earthquakes, several reasons were reported about the cause of the damages including non-ductile construction details, strong beams-weak columns, short columns, heavy overhangs, lack of quality control and maintenance, and substandard old code requirements [2, 5-13].

Albania has been struck by several strong ground motions which caused a lot of losses in human life as well as property [3, 14-15]. Significant seismic events over last century, are summarized in Table 1. The large number of recorded deaths and severely damaged and collapsed buildings in Albania has highlighted again the insufficient performance of RC buildings in the region and in other countries which have similar construction practices. The large number of losses of lives and property destructions were caused by the collapse of seriously damaged or collapsed of usually four to six stories high reinforced concrete buildings, during the November 26, 2019 earthquake.

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Table 1. Major tremors in Albania [3, 15]

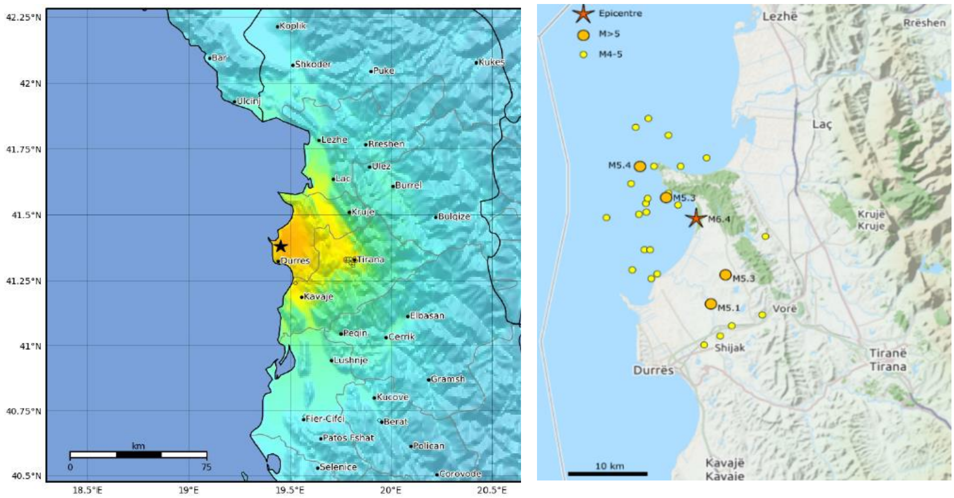
Date	Impacted region	Magnitude (M_w)	Depth (km)	Consequences	
				Dead	Injured
26-Nov-2019	Durres	6.4	20.0	51	Over 3000
21-Sep-2019	Durres	5.6	10.0	-	108
9-Jan-1988	Tirana	5.4	24.0	-	-
16-Dec-1982	Fier	5.6	21.9	1	12
15-Apr-1979	Shkoder; Montenegro	6.9	10.0	136	Over 1000
30-Nov-1967	Diber	6.6	20.0	12	174
18-Mar-1962	Fier	6.0	-	5	77
26-May-1960	Korçe	6.4	-	7	127
1-Sep-1959	Fier	6.2	20.0	2	-
27-Aug-1948	Shkoder	5.5		1	27
27-Aug-1942	Diber	6.0	33.0	43	110
21-Nov-1930	Vlore	6.0	35.0	30	100
26-Nov-1920	Tepelena	6.4	-	36	102
22-Dec-1919	Leskovik;Konica	6.1	-	-	-
6-Jan-1905	Shkoder	6.6	-	200	500

The earthquake performance of residential masonry structures has been questioned in several studies after the example of November 26, 2019 earthquakes [11, 16-21]. On the other hand, it is important to highlight that a large number of the reinforced concrete buildings damaged during this earthquake sequences, were residential buildings constructed per pre-modern codes of practice.

In Albania, template designs developed by the governmental authorities are used for many of the buildings planned both for residential and public services as a common practice to save architectural fees and ensure quality control during communist era, till 1990s. There are standard RC framed buildings from 4-6 stories constructed according to the older code requirements [22-24]. The target of this study aims to assess the seismic performance of the reinforced concrete residential buildings built per premodern seismic code requirements [22] in Albania considering the inelastic response of RC components. Four buildings having template designs were chosen to represent the commonly used RC residential buildings in high seismic regions of the country. Selection of the building typologies and material properties were determined based on the site investigations and archive studies after November 26, 2019 earthquake sequences in Durres city. The capacity curves of the typologies investigated, were calculated using nonlinear static analyses in both directions. The nonlinear dynamic characteristics were simulated by ESDOF system approach. Earthquake displacement demands were estimated under a set of far-fault and near-fault ground motions. The probability of failure was comparatively estimated under the selected group of records for each typology. Reasons of the damages in the recent earthquakes were discussed using the results of performance evaluation of the selected buildings.

1.2. Albanian Earthquakes During 2019 and Its Consequences to RC Buildings

On Saturday, September 21, 2019, at 15:15 CET, an earthquake with a shallow depth [17] and a moment magnitude of Mw = 5.6 hit the northwestern region of Albania, as shown in Fig. 1. The epicenter of the earthquake was close to Durrës city. Regardless of being close to the city, the incident had relatively slight effects without fatalities, causing non-structural damage to the buildings [18]. After three months, on 26 November at 03:54 Central European Time (CET), central and north-western Albania was struck by the main shock of the earthquake (Mw = 6.4) series.



(a) (b)

Fig. 1 Epicentral locations for 2019 Albania earthquakes: (a) September 21, (b) November 26, Durrës earthquakes (USGS, 2019)

This second strong earthquake proved the high seismic vulnerability of reinforced concrete buildings as shown in Fig. 2. The epicenter of the earthquake was located in Durrës (northwest Albanian region) and 30-km west of the capital city (Tirana). The mainshock occurred at 03:54 CET (UTC+1), at a relatively shallow depth of about 16 km, with a magnitude of Mw = 6.4 [18]. The horizontal peak ground acceleration measured in Durres was approximately 0.20 g, and in Tirana, this value was about 0.12 g [25]. The Durrës station had an electricity cut after first 15 seconds of measuring the record, hence 0.20 g value received from this station should be considered as a lower limit of the actual peak ground acceleration occurred in the center.

The seismic activity extensively impacted the biggest municipalities of the region and damaged more than 14,000 buildings in densely populated cities, including 51 deaths, over 3,000 wounded people and more than 14,000 remained homeless [26]. Therefore, the proximity of the major fault to the city of Tirana and Durrës triggered serious damage or partial collapse of some buildings, resulting in massive damage to both old and newly designed RC buildings and in loss of lives. As shown in Fig. 2, most of the investigated buildings suffered from weak concrete strength, material aging, inappropriate reinforce detailing, poor workmanship, corrosion of steel bars, in-plane and out-plane failures [27].

Thanks to the close collaboration with Albanian Construction Institute (ISTN) representatives and practicing engineers for the damage assessment on the earthquake affected area. Based on the findings, Table 2 outlines damage levels of the investigated buildings [28].

1.3. Current Seismic Design Code in Albania

KTP-N.2 (1989) was published as an update of KTP 2 (1978) and is still the official Albanian seismic code. It provides design provisions of a wide range of structural configurations. The seismic hazard is defined by macro-seismic intensity areas which are defined according to MSK-64 scale. The country is divided into three large seismic zones with intensity VI, VII, VIII. KTP-N.2-89 defines the seismic design actions considering the load combinations and the influence of torsional effects. The analysis methods include time history and response spectrum for which the horizontal design acceleration is calculated using:

$$S_{a(T)} = k_E \, k_r \, \psi \, \beta \, g \tag{1}$$

where "k_E" is the seismic coefficient, "k_r" is the importance factor, "ψ" – importance coefficient, "β" – dynamic coefficient and "g" is the gravitational acceleration.

Fig. 2 Consequences to reinforced concrete buildings during the 2019 Albania earthquakes

Table 2. Damage assessment results after the earthquake on 26 November 2019 after the earthquake [28]

City/Damage levels	No damage	Damage Limitation			Significant damage	Near Collapse	Total
DURRËS	22605	2761	2384	1735	1855	626	31966
LEZHE	494	364	421	326	402	43	2050
TIRANE	5651	1560	1258	737	974	386	10566
TOTAL	28750	4685	4063	2798	3231	1055	44582

The code considers several lateral resisting systems such as dual systems, frames with masonry infills and moment-resisting frames for reinforced concrete buildings. In general, KTP-N.2 (1989) lacks many detailing recommendations, even though it shares similar principles with modern seismic design codes such as Eurocode 8. The code mostly intends to protect the structure from collapse rather than giving sufficient recommendations about damage limitations. On the other hand, it requires that columns must be designed to withstand more forces than beams "Strong-Column Weak-Beam" but does not provide sufficient information to perform the necessary checks.

2. Description of the Studied Buildings

Albanian building stock is mostly composed of prefabricated reinforced concrete, brick and stones, wood and other construction materials [29]. According to Albanian Institute of Statistics INSTAT 2011 [30], 85% of the total residential building stock are composed of one-story buildings. This category includes unreinforced masonry URM, clay masonry CM buildings made of stones, clay or silicate and reinforced concrete RC frames with masonry infills. Generally, the roof of the buildings falling in this category, is made of wood trusses. Table 3 provides information regarding the year of construction based on different materials.

Table 3. Classification of the Albanian building stock [1]

Material of the construction	Before 1945	1945-1960	1961-1980	1981-1990	1991-1995
Prefabricated concrete	0	0	4601	5993	4575
Masonry	37,416	63,870	141,174	102,198	43,324
Timber	462	-	1,821	1,273	7,43
Others	2,560	3,393	7,105	6,263	4,238
Total	40,438	67,263	154,701	115,727	52,880

A site survey was carried out in Durres and Tirana cities to select the commonly encountered typologies among the residential buildings. As an important center for tourism and export, Durres represents a mid-size city in the Albanian earthquake zone [25]. After November 26. 2019 earthquake, authors made several visits to the earthquake-stricken area to investigate the reasons of the damages on the environment. It was observed that the most used templates for residential RC building typologies are Template Design Buildings (TD). An example falling into this category and taken during the investigations, is shown in Fig. 3.

In this study there are considered four buildings and labeled as TD_1, TD_2, TD_3 and TD_4. Each of the template designs is reinforced concrete (RC) residential building having no shear wall in any direction. They were designed in 1982 according to KTP 2-78 and are still in use nowadays. Each of the designs fall into the category of midrise buildings, TD_1 and TD_2 is 5-story and TD_3 and TD_4, 6-story. The maximum height of TD_1 and TD_2 reaches 14.42m considering the parapet used in the roof story as shown in Fig. 4 and Fig. 5. The structural plan of first three buildings is a regular one. TD_2 and TD_3 have the same planimetry which has one more frame than TD_1. However, TD_3 (Fig. 6) has 6 stories and reaches the maximum height of 17.22m. The last building, TD_4 is 17.67 m long and has an irregular plan as shown in Fig. 7. All buildings have a typical story height of 2.8m except the last one which tends to be higher slightly differing from each other. Representative plan views of the selected buildings are given in the Figures 4-7.

Fig. 3 Location of the damaged template design building considered in this study with respect to epicenter of November 26, 2019 earthquake.

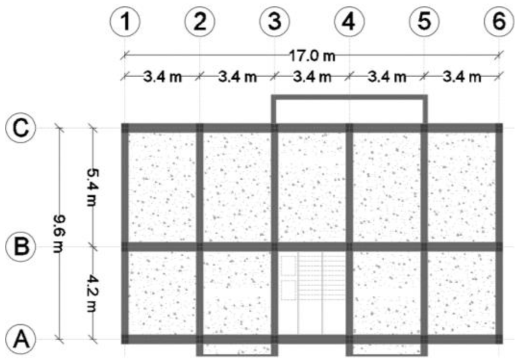


Fig. 4 The plan view of the Template Design 1 (TD_1) building

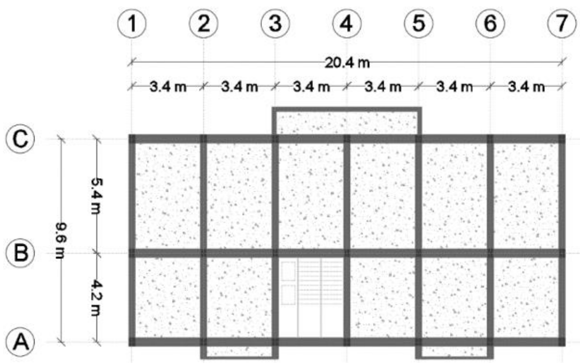


Fig. 5 The plan view of the Template Design 2 (TD_2) building

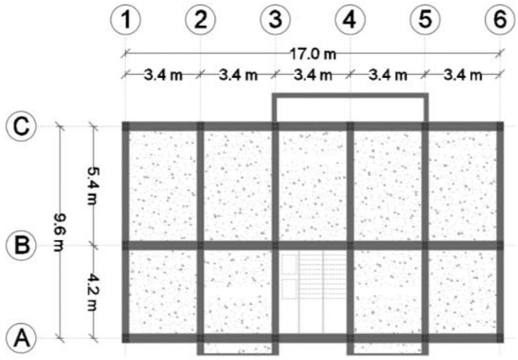


Fig. 6 The plan view of the Template Design 3 (TD_3) building

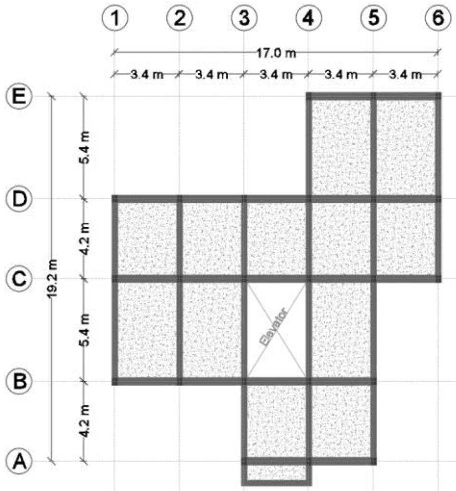


Fig. 7 The plan view of the Template Design 4 (TD_4) building

The residential typologies considered in this study are reinforced concrete (RC) moment resisting framed buildings in both transverse and longitudinal directions. Table 4 lists the summary of the buildings selected.

Table 4. Summary of the typologies selected

Properties	Template design identifications			
	TD-1	TD-2	TD-3	TD-4
Floor area: m^2	172.39	221.47	221.47	208.51
# of stories	5	5	6	6
Structural type	Reinforced Concrete Frame			
Typical beam dimensions (cm)	30x40	30x40	30x40	40x20
Typical column dimensions (cm)	30x40	30x40	30x40	30x40

Template designed buildings generally have uniform distribution of mass and stiffness in both horizontal and vertical planes due to architectural similarities and purpose of use in all stories. Therefore, they are not subjected to structural irregularities. All the selected typologies have symmetrical or close to symmetrical layouts in both principal directions, except TD_4. One of the potential major shortcomings of these typologies per pre-modern Albanian Code is the strong-beam weak-column response due to the lack of attention. Since there are no shear walls in their lateral load bearing systems, formation of plastic hinges in columns may remarkably affect overall response, causing loss of lateral stiffness in a single floor. This deficiency seems to be a critical weakness among these typologies.

Typical longitudinal rebar details for beams and columns are given in Fig. 8. The symbol 'Ø' is placed after the number of rebars, and shows the diameter of it in mm. Fig. 8 shows both beams and columns have low amount of longitudinal rebars, for TD_1-3 generally around 1.05% and 0.77% and for TD_4 around 1.27% and 0.45% of cross-sectional area for columns and beams, respectively.

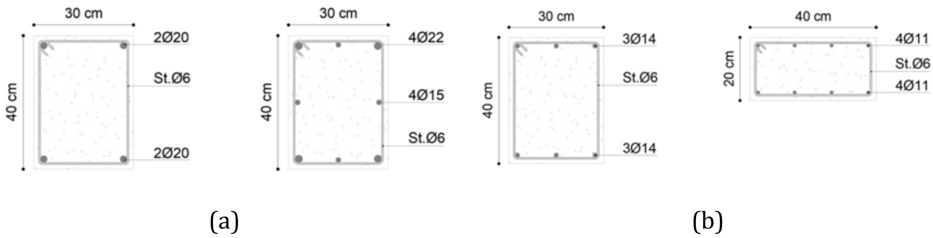


Fig. 8 Typical column and beam details: (a) Typical column for TD_1, TD_2, TD_3 (left) and TD_4 (right) and (b) Typical neam for TD_1, TD_2, TD_3 (left) and TD_4 (right)

Typical transverse reinforcement given in design drawings for columns is Ø6 with 200 mm spacing for TD_1-3 and Ø6 with 150 mm spacing for TD_4. For beams the transverse reinforcement and stirrups spacing remains the same for all template designs as Ø6 with 250 mm spacing.

3. Material Characteristics

For the analytical modelling of the selected residential buildings, material properties obtained from experimental tests and site investigations were considered. As discussed before, template buildings intended for residential purposes have similar design procedures controlled by governmental authorities at the time of their construction.

Concrete and steel specimens were extracted, and experimental tests were performed on one of the selected building typologies (TD_2 - 82/2). Based on the blueprints' details, the concrete class and the steel grades were determined for each of the buildings. Referring to each of the template designs, the concrete class M200 (C16/20) is used. For the reinforcement, 2100 Kg/cm2 (Ç-3) steel material is used in the design. Table 5 and Table 6 summarize the detailed properties of the reinforcement steel and concrete respectively. Moreover, the laboratory tests for concrete sample, is shown in Fig. 9.

Moreover, concrete and steel specimens were taken to investigate the inherent characteristics of the building's material. Based on the findings of these tests, compressive strength of the concrete samples was found to be about half of the design requirements of the Albanian design code (KTP.N2.89) as shown in the Table 7.
On the other hand, test results on steel specimens shown that they are acceptable according to the design definitions, Table 8.

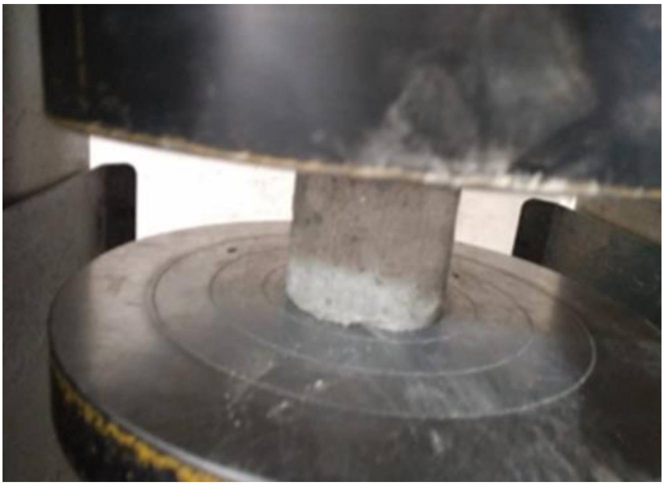


Fig. 9 Laboratory test for the concrete sample of TD_2 - 82/2

Table 5. Properties for steel material "Ç-3"

Properties of Steel Material	"Ç-3"
Tensile strength	f_{ck} = 250 MPa
Yield strength	f_{yk} = 320 MPa
Young's Modulus	E_s = 210 GPa
Partial factor	γ_s = 1.15
Design yield (shear)	f_{ywd} = 180 MPa
Design yield (strength)	f_{yd} = 215 MPa
Poisson's ratio	ν = 0.30

Table 6. Properties for concrete C16/20

Properties of Concrete	C16/20
Cubic strength	f_{ck} = 16 MPa (fc,cube)
Compressive cylinder strength	f_{ck} = 20 MPa
Mean value of cylinder compressive strength (28 days)	f_{cm} = 28 MPa
Characteristic axial tensile strength	$f_{ctk(95\%)}$ = 2.9 MPa
Characteristic axial tensile strength	$f_{ctk(5\%)}$ = 1.5 MPa
Mean value of axial tensile strength	f_{ctm} = 2.2 MPa
Young's Modulus	E_{cm} = 30 GPa
Design value of modulus of elasticity	E_{cd} = 25 GPa
Design value of compressive strength	$f_{cd} = {}^{\alpha * fck}/_{\gamma c}$ = 11.3 MPa
Partial factor	γ_c = 1.5 and α = 0.85
Poisson's ratio	ν = 0.20

Table 7. Laboratory results for concrete

Sample (Nr.)	K1	K2
Sample height (H)	77.0	77.5
Sample diameter (D)	75.0	75.0
H/D ratio	1.03	1.03
Weight (g)	778	797
Density (g/cm^3)	2.287	2.328
Load (kN)	27.9	35.6
Compression Strength (MPa)	6.32	8.06

Table 8. Laboratory results for steel

Sample (Nr.)	1	2	3
Nominal Diameter (mm)	14	16	22
Measured Diameter (mm)	14.96	15.87	21.91
Linear weight (kg/m)	1.377	1552	2.958
Cross-sectional area (mm^2)	175.4	197.66	376.75
Tensile strength (N/mm^2)$б_y$	267.6	269.4	331.8
Ultimate strength (N/mm^2)$б_u$	402.1	400.2	469.4
$б_u$/$б_y$ Ratio	1.502	1.486	1.415
Relative Deformation (%)	32.14	35.00	30.00

4. Mathematical modelling

Member dimensions and reinforcements in the typical designs were used to develop the analytical models of the selected buildings for inelastic analysis. All components were modelled as given in their respective designs according to the project details.

Nonlinear static analyses (Pushover) are simulated using a finite element software (Zeus_NL) which is established especially for earthquake engineering applications [31, 32]. Zeus NL has the ability to monitor the cross-section in different fibers such as: confined fiber, unconfined fiber and reinforcement by utilized a fiber approach for the nonlinear analysis by as shown in Fig. 10. Case study buildings are modelled as 2D moment-frame using the middle frames for longitudinal and transverse directions. From the software library it is selected the cubic elasto-plastic type 3D option to determine the structural elements for the building models considered in this study. For the steel reinforcement it is used a bilinear elasto-plastic material model which considered the kinematic strain hardening (stl1). Whereas for the concrete was used the uniaxial constant confinement concrete material model (conc2).

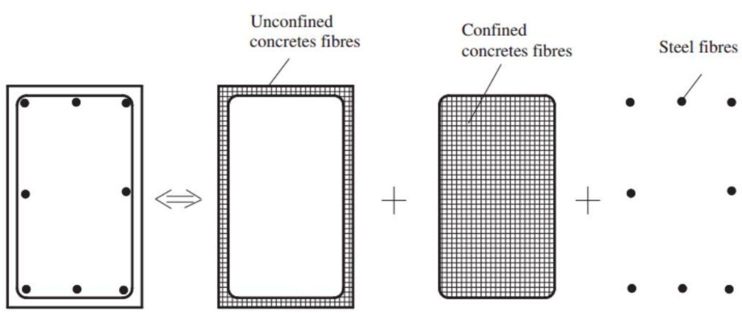


Fig. 10 Decomposition of a RC rectangular section (A. Elnashai et. al., 2002).

Program updates the section properties under different loading conditions, hence material properties for the structural elements belong to the uncracked ones. Capacity curves are developed under a reverse triangular loading pattern applied laterally together with gravitational loads from story mass. Pushover graphs are plotted in horizontal axis by the ratio of displacement of the roof story and building height, whereas in vertical axis, by the ratio of base shear and total weight of the building.

4.1. Ground Motions

The selection of ground motion records is a crucial step in nonlinear time history analyses because the use of acceleration records with the same characteristics may underestimate or overestimate the building response. For this study there were used 46 far-fault and 54 near-fault ground motions recorded in dense-hard ground areas to investigate the effect of far and near-fault earthquakes on the seismic behavior of the selected template designs. Table 9 and Table 10 lists the main characteristics of the records considered in this study.

Table 9. Far-fault ground motion dataset

Nr	Earthquake	Record and component	Year	M_w	Site	d(km)	PGD (cm)	PGV (cm/s)	PGA (g)
1	San Fernando	LA HOLLYWOOD STOR LOT (90)	1971	6.6	C	62.2	12.42	18.93	0.21
2	San Fernando	LA HOLLYWOOD STOR LOT (180)	1971	6.6	C	62.2	6.32	14.87	0.17
3	Friuli, Italy	TOLMEZZO (0)	1976	6.5	C	37.7	4.11	22.03	0.35
4	Friuli, Italy	TOLMEZZO (270)	1976	6.5	C	37.7	5.09	30.80	0.32
5	Imperial Valley	DELTA (262)	1979	6.9	D	43.6	11.99	26.00	0.24
6	Imperial Valley	DELTA (352)	1979	6.9	D	43.6	19.03	33.02	0.35
7	Imperial Valley	EL CENTRO ARRAY #11 (140)	1979	5.2	D	30.3	16.08	34.44	0.36
8	Imperial Valley	EL CENTRO ARRAY #11 (230)	1979	5.2	D	30.3	18.63	42.14	0.38
9	Superstition Hills	EL CENTRO IMP CO CENTER (0)	1987	6.5	B	18.5	17.53	46.36	0.36
10	Superstition Hills	EL CENTRO IMP CO CENTER (90)	1987	6.5	B	18.5	20.10	40.87	0.26
11	Superstition Hills	POE (270)	1987	6.5	B	14.7	8.82	35.80	0.45
12	Superstition Hills	POE (360)	1987	6.5	B	14.7	11.28	32.80	0.30
13	Loma Prieta	CAPITOLA (0)	1989	7.1			9.13	35.01	0.53
14	Loma Prieta	CAPITOLA (90)	1989	7.1			5.49	29.21	0.44
15	Loma Prieta	GILROY ARRAY #3 (0)	1989	7.1	D	14.4	8.26	35.69	0.56
16	Loma Prieta	GILROY ARRAY #3 (90)	1989	7.1	D	14.4	19.33	44.67	0.37
17	Cape Mendocino	RIO DELL OVERPASS FF (360)	1992	7.0	D	18.5	19.55	42.00	0.55
18	Cape Mendocino	RIO DELL OVERPASS FF (270)	1992	7.0	D	18.5	7.02	10.54	0.20
19	Landers	COOLWATER (LN)	1992	7.3	C	69.2	13.71	25.64	0.28
20	Landers	COOLWATER (TR)	1992	7.3	C	69.2	13.81	42.34	0.42
21	Landers	YERMO FIRE STATION (270)	1992	7.3	D	23.6	43.85	51.44	0.25
22	Landers	YERMO FIRE STATION (360)	1992	7.3	D	23.6	24.63	29.71	0.15
23	Northridge	BEVERLY HILLS - 12520 MULH (35)	1994	6.7			8.57	40.86	0.62

Table 1 (Con). Far-fault ground motion dataset

Nr	Earthquake	Record and component	Year	Mw	Site	d(km)	PGD (cm)	PGV (cm/s)	PGA (g)
24	Northridge	BEVERLY HILLS - 12520 MULH (125)	1994	6.7			4.83	30.19	0.44
25	Northridge	BEVERLY HILLS - 14145 MULH (9)	1994	6.7	C	19.6	13.15	58.94	0.42
26	Northridge	BEVERLY HILLS - 14145 MULH (279)	1994	6.7	C	19.6	11.07	62.78	0.52
27	Northridge	CANYON COUNTRY - W LOST CANYON (0)	1994	6.7	D	13.0	11.71	43.03	0.41
28	Northridge	CANYON COUNTRY - W LOST CANYON (270)	1994	6.7	D	13.0	12.54	45.38	0.48
29	Kobe	NISHI-AKASHI (0)	1995	6.9	D	22.5	9.53	37.29	0.51
30	Kobe	NISHI-AKASHI (90)	1995	6.9	D	22.5	11.26	36.67	0.50
31	Kobe	SHIN-OSAKA (0)	1995	6.9	D	19.2	8.55	37.86	0.24
32	Kobe	SHIN-OSAKA (90)	1995	6.9	D	19.2	7.64	27.94	0.21
33	Kocaeli	ARCELIK (0)	1999	7.4	C	17.0	13.65	17.69	0.22
34	Kocaeli	ARCELIK (90)	1999	7.4	C	17.0	35.58	39.55	0.15
35	Kocaeli	DUZCE (180)	1999	7.4	D	17.1	44.13	58.88	0.31
36	Kocaeli	DUZCE (270)	1999	7.4	D	17.1	17.62	46.39	0.36
37	Chi-Chi	CHY101 (E)	1999	7.6	D	11.1	45.30	70.64	0.35
38	Chi-Chi	CHY101 (N)	1999	7.6	D	11.1	68.76	115.00	0.44
39	Chi-Chi	TCU045 (E)	1999	7.6	C	26.0	50.68	36.70	0.47
40	Chi-Chi	TCU045 (N)	1999	7.6	C	26.0	14.35	39.09	0.51
41	Duzce	BOLU (0)	1999	7.1	D	12.0	23.07	56.49	0.73
42	Duzce	BOLU (90)	1999	7.1	D	12.0	13.56	62.12	0.82
43	Iran_Manjil	LONGITUDINAL COMP	1990	7.4	-	74.0	14.92	43.26	0.52
44	Iran_Manjil	TRANSVERSE COMP	1990	7.4	-	74.0	20.83	55.55	0.50
45	Hector Mine	HEC (0)	1999	7.1	-	22.0	22.54	28.58	0.27
46	Hector Mine	HEC (90)	1999	7.1	-	22.0	13.96	41.75	0.34

Table 10. Near-fault ground motion dataset

Nr	Earthquake	Record and component	Year	M_w	Site	d(km)	PGD (cm)	PGV (cm/s)	PGA (g)
1	Imperial Valley	CHIHUAHUA (12)	1979	6.5		-	9.13	24.85	0.27
2	Imperial Valley	CHIHUAHUA (282)	1979	6.5		-	12.91	30.12	0.254
3	Imperial Valley	EL CENTRO ARRAY #6 (140)	1979	6.5	D	1	27.57	64.83	0.41
4	Imperial Valley	EL CENTRO ARRAY #6 (230)	1979	6.5	D	1	65.82	109.8	0.439
5	Imperial Valley	EL CENTRO ARRAY #7 (140)	1979	6.5	D	0.6	24.65	47.6	0.338
6	Imperial Valley	EL CENTRO ARRAY #7 (230)	1979	6.5	D	0.6	44.71	109.24	0.463
7	Imperial Valley	BONDS CORNER (140)	1979	6.5	D	2.5	0.34	3.61	0.084

Table 2 (Cont). Near-fault ground motion dataset

Nr	Earthquake	Record and component	Year	M_w	Si te	d(k m)	PGD (cm)	PGV (cm/s)	PGA (g)
8	Imperial Valley	BONDS CORNER (230)	1979	6.5	D	2.5	1.42	8.18	0.1
9	Irpinia Eq / Italy	STURNO (0)	1980	6.9	C	10.8	11.58	36.39	0.251
10	Irpinia Eq / Italy	STURNO (270)	1980	6.9	C	10.8	32.02	51.82	0.358
11	Nahanni, Canada	SITE 1 (10)	1985	6.8	B	6	9.64	46.05	0.978
12	Nahanni, Canada	SITE 1 (280)	1985	6.8	B	6	14.52	46.13	1.096
13	Nahanni, Canada	SITE 2 (240)	1985	6.8	B	6	7.54	29.26	0.489
14	Nahanni, Canada	SITE 2 (330)	1985	6.8	B	6	6.57	33.13	0.323
15	Superstition Hills	PTS (225)	1987	6.6	D	0.7	52.83	112	0.455
16	Superstition Hills	PTS (315)	1987	6.6	D	0.7	15.25	43.9	0.377
17	Loma Prieta	BRAN (0)	1989	6.9			11.69	55.74	0.481
18	Loma Prieta	BRAN (90)	1989	6.9			11.86	41.91	0.526
19	Loma Prieta	CORRALITOS (0)	1989	5.1	D	5.1	10.82	55.16	0.644
20	Loma Prieta	CORRALITOS (90)	1989	5.1	D	5.1	11.29	45.5	0.479
21	Loma Prieta	SARATOGA ALOHA AVE (0)	1989	6.9	C	4.1	16.24	51.15	0.512
22	Loma Prieta	SARATOGA ALOHA AVE (90)	1989	6.9	C	4.1	27.61	42.61	0.324
23	Erzican / Turkey	ERZICAN EAST-WEST COMP ()	1992	6.7	D	4.4	21.92	64.3	0.496
24	Erzican / Turkey	ERZICAN - NORTH-SOUTH COMP ()	1992	6.7	D	4.4	27.66	83.95	0.515
25	Cape Mendocino	CAPE MENDOCINO (0)	1992	7.1	B	9.5	39.74	125.57	1.497
26	Cape Mendocino	CAPE MENDOCINO (90)	1992	7.1	B	9.5	12.18	41.33	1.039
27	Cape Mendocino	PETROLIA (0)	1992	7.1	B	9.5	21.97	48.32	0.59
28	Cape Mendocino	PETROLIA (90)	1992	7.1	B	9.5	29.01	90.08	0.662
29	Landers	LUCERNE (260)	1992	7.3	B	2	217.12	146.03	0.727
30	Landers	LUCERNE (345)	1992	7.3	B	2	52.78	32.94	0.789
31	Northridge Earthquake	CA:LA;SEPULVEDA VA (BLD 40 GND; 270)	1994	6.7	D	9.5	13.39	78.1	0.749
32	Northridge Earthquake	CA:LA;SEPULVEDA VA (BLD 40 GND; 360)	1994	6.7	D	9.5	17.39	76.15	0.934
33	Northridge Earthquake	NORTHRIDGE - SATICOY (90)	1994	6.7	D	13.3	8.44	28.96	0.368
34	Northridge Earthquake	NORTHRIDGE - SATICOY (180)	1994	6.7	D	13.3	22.07	61.46	0.477
35	Northridge Earthquake	RINALDI RECEIVING STA (228)	1994	6.7	D	8.6	29.62	160.33	0.825
36	Northridge Earthquake	RINALDI RECEIVING STA (318)	1994	6.7	D	8.6	26.96	74.54	0.487
37	Northridge Earthquake	SYLMAR - HOSPITAL (90)	1994	6.7	D	6.4	16.82	78.37	0.604
38	Northridge Earthquake	SYLMAR - HOSPITAL (360)	1994	6.7	D	6.4	31.96	130.4	0.843
39	Kocaeli / Turkey	IZMIT (90)	1999	7.4	B	4.3	17.13	29.78	0.22
40	Kocaeli / Turkey	IZMIT (180)	1999	7.4	B	4.3	9.81	22.61	0.152

Table 3 (Cont). Near-fault ground motion dataset

Nr	Earthquake	Record and component	Year	M_w	Site	d(km)	PGD (cm)	PGV (cm/s)	PGA (g)
41	Kocaeli / Turkey	YARIMCA (330)	1999	7.4	D	3.3	50.98	62.16	0.349
42	Kocaeli / Turkey	YARIMCA (60)	1999	7.4	D	3.3	57.03	65.72	0.268
43	Chi-Chi	TCU065 (E)	1999	7.6	D	2.5	92.59	126.18	0.814
44	Chi-Chi	TCU065 (N)	1999	7.6	D	2.5	60.75	78.79	0.603
45	Chi-Chi	TCU067 (E)	1999	7.6	D	1.1	93.12	79.58	0.503
46	Chi-Chi	TCU067 (N)	1999	7.6	D	1.1	45.96	66.7	0.325
47	Chi-Chi	TCU084 (E)	1999	7.6	C	11.4	31.44	114.74	1.157
48	Chi-Chi	TCU084 (N)	1999	7.6	C	11.4	21.27	45.58	0.417
49	Chi-Chi	TCU102 (E)	1999	7.6	D	1.2	89.2	112.45	0.298
50	Chi-Chi	TCU102 (N)	1999	7.6	D	1.2	44.88	77.16	0.169
51	Duzce	DUZCE (180)	1999	7.4	D	11	42.11	59.97	0.348
52	Duzce	DUZCE (270)	1999	7.4	D	11	51.62	83.49	0.535
53	Denali Alaska	PS10 (47)	2002	7.9	D	5	102.73	134.73	0.319
54	Denali Alaska	PS10 (317)	2002	7.9	D	5	77.99	75.97	0.318

4.2. Pushover Analysis

This analysis consists of the application of a representative lateral load pattern together with the gravity load effects. In each case, monotonically increased lateral loads, which were proportional with the product of the first mode shape and mass, were applied to obtain capacity curves of the selected buildings. P-Δ effects were considered during the analyses. The response of the buildings is simulated by capacity (pushover) curves where the variation of roof displacement is plotted with respect to base shear force. This representation is useful for practicing engineers.

Pushover curves of each building was obtained for various concrete strengths and stirrups spacings commonly encountered from the site visits after November 26, 2019 earthquakes; four concrete strength and four transverse reinforcement spacing values were considered. The analyses of four buildings in two orthogonal directions resulted in 128 pushover curves. The notation in the tables and figures corresponds to concrete strength in MPa and stirrups spacings in mm. For example, the C16-S100 means that the building with 16 MPa concrete strength (C16) and 100 mm stirrups spacing (S100). This following part gives a brief summary of the capacity curves assessment.
To better understand the boundaries of behavior for typical residential buildings, two extreme cases were considered from the template designs: average (C16-S100) and (C10-S250) poor construction quality. Pushover curves representing the average and poor conditions are shown in Fig. 11 and Fig. 12 for both orthogonal directions.

4.3. Strength and Deformation Capacities

Using the capacity curves, performance assessment of the investigated residential buildings was done. Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) are considered in this study as stated in many international guidelines [33-36]. Pushover analyses outputs were used to obtain global drift capacities of each template design Table 11.

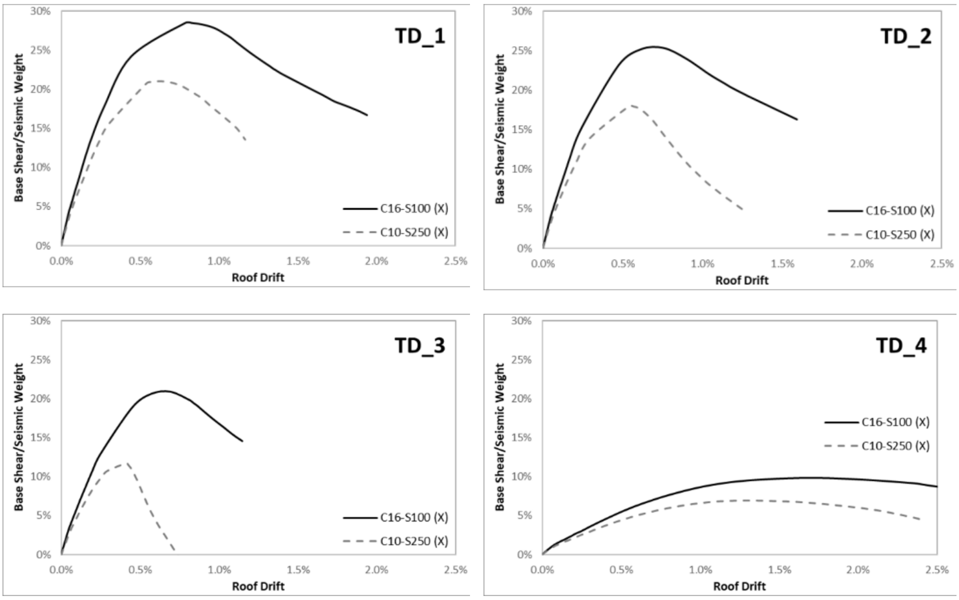


Fig. 11 Pushover curves of the buildings in weak and average conditions in longitudinal direction (x direction)

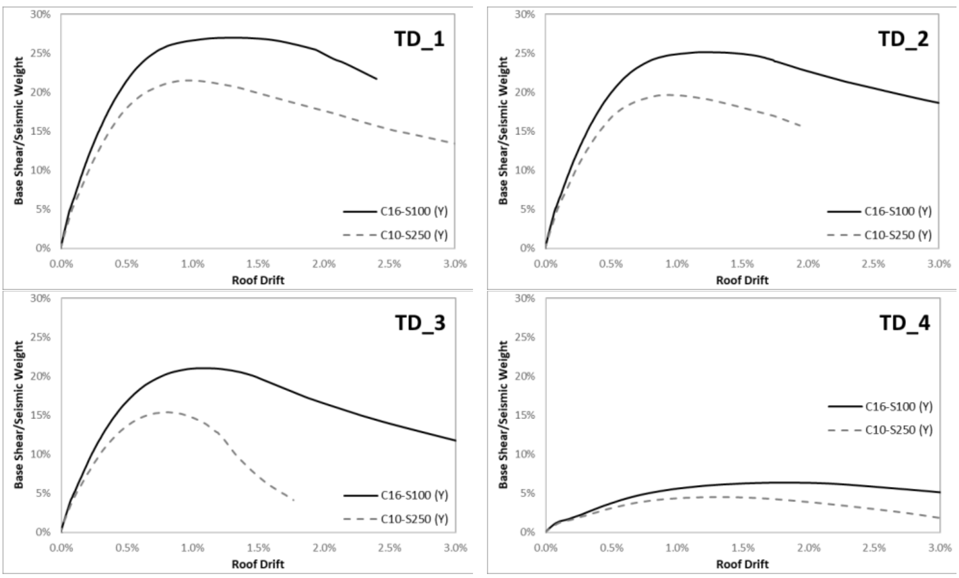


Fig. 12 Pushover curves of the buildings in weak and average conditions in transverse direction (y direction)

Pushover curves for each template designs are shown in Fig. 11 and Fig. 12. Considerably small displacement capacities are noteworthy since the buildings' response are dominated by the frame action.

The effects of transverse reinforcement spacing and concrete quality on drift and lateral load bearing capacity are clearly seen in Fig.11-12 and Table 11. As shown in Table 11 there is a considerable drop in both drift and base shear ratio from C16 to C10 and from

S100 to S250 stirrups spacing. The average amount of reduction for all buildings is around 30% from models designed with C16-S100 to C10-S250. Table 12 gives a detailed information on the performance reduction of the buildings as an influence of these two important factors.

Table 11. Displacement capacities (%) of the selected residential buildings obtained from pushover curves for the considered performance levels

Template Design ID	Material Quality	X-direction			Y-direction		
		IO	LS	CP	IO	LS	CP
		$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$	$\Delta_{roof}/H_{building}$
TD_1	C10-S250	0.27	0.61	0.90	0.56	1.21	1.85
	C16-S100	0.23	0.73	1.23	0.57	1.41	2.76
TD_2	C10-S250	0.25	0.49	0.72	0.53	1.16	1.79
	C16-S100	0.20	0.68	1.17	0.60	1.48	2.36
TD_3	C10-S250	0.28	0.38	0.47	0.55	0.86	1.16
	C16-S100	0.22	0.58	0.94	0.63	1.47	2.07
TD_4	C10-S250	0.92	1.48	2.04	0.76	1.39	2.02
	C16-S100	1.05	1.84	2.75	0.88	1.81	2.80

Table 12. Drift and Lateral load bearing rations for different concrete quality and stirrups spacing in percentage.

		C10-S250 (X)	C16-S100 (X)	C10-S250 (Y)	C16-S100 (Y)
TD_1	Base shear ratio	21.0%	28.5%	21.5%	27.0%
	Drift ratio	1.0%	1.3%	2.1%	2.4%
TD_2	Base shear ratio	18.0%	25.4%	19.6%	25.1%
	Drift ratio	0.7%	1.2%	1.9%	2.6%
TD_3	Base shear ratio	11.8%	20.9%	15.4%	21.0%
	Drift ratio	0.5%	1.0%	1.2%	1.9%
TD_4	Base shear ratio	6.9%	9.9%	4.5%	6.4%
	Drift ratio	2.1%	2.7%	2.1%	2.9%

A careful consideration of Fig. 11 and Fig. 12 together with Table 11 reveals that the yield base shear rate and global drift capacity, especially at the CP performance level, appear to differ significantly from those found in the relevant literature (i.e. JICA, HAZUS) [37, 38]. This difference might be due to the code enforcements, construction practice and possibly the influence of modelling strategy. Another important observation for the low displacement capacities is related with the failure mechanisms of the buildings since the pre-modern code (KTP-78) requirements did not consider the weak-beam strong-column formation which has been a common problem for building construction practice of Albania or similar countries.

4.4. Nonlinear Time History Analyses for Seismic Demand Estimations

The pushover curve for each template design is approximated with a bilinear curve by using the outlined criteria in the relevant literature [33, 34]. Yield point on the pushover curve is defined as the point where the structure starts to soften. A sample of capacity and idealized pushover curve is shown in Fig. 13. Yield and ultimate behavior points represent the bi-linearized pushover curve.

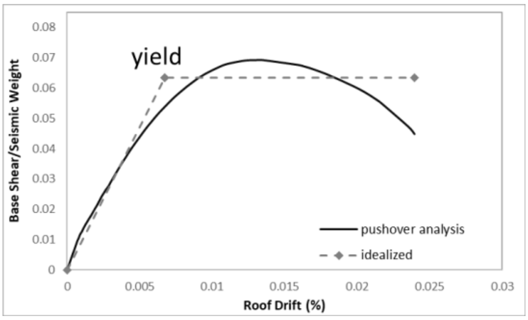


Fig. 13 A sample capacity and idealized pushover curve

International guidelines such as ATC 40 or FEMA 356, provide information for illustration of ESDOF of building capacity curve. In this study, ATC-40 was used for the representation of the ESDOF response. Below there are presented the equations for the yield displacement (Δ_y) and yield strength (C_y) coefficients:

$$\Delta_y = \frac{\Delta_{y,roof}}{\Gamma_1} \tag{2}$$

$$C_y = \frac{S_a}{g} = \frac{V_{y,mdof}/W}{\alpha_1} \tag{3}$$

The ESDOF models of each RC building were subjected to ground motion listed in Table 9-10 to estimate the displacement demands. Nonlinear response history analyses were carried out using a computer program for Nonlinear Dynamic Time History Analysis of Single and Multi-Degree of Freedom Systems, (Nonlin 8.0)" [39]. Next, the displacement demands were converted accordingly for the roof top considering first mode participation factor. In addition to the demand calculation, the seismic performance evaluation of each template design building was conducted using the set of the records listed in Table 9-10.

5. Discussion of the Results and Conclusions

The average exceedance-ratio of the estimated limit states is summarized in Table 13 for near-fault and far-fault ground motions. For the performance evaluation, with a moderate exceedance ratio, the performance level is satisfied if the average exceedance rate is less than 0.5. As can be seen in Table 13, the Immediate Occupancy (IO) performance point is exceeded in most buildings. Life Safety (LS), similar to Immediate Occupancy, shows the same trend for most of the cases as well. Although the Collapse Prevention (CP) is not required in residential buildings, it is an important factor for limiting injuries and preventing loss of life during an earthquake. The exceedance ratio for the Collapse Prevention CP performance level reaches 0.66. Table 13 clearly shows that the effects of near fault in reaction to reinforced concrete residential properties are significant for each performance level. Moreover, Table 13 clearly shows that existing template designs are not even close to satisfy the Immediate Occupancy limit state during earthquakes that may have a similar effect to selected records. In addition, more than half of the existing structures are at a critical level of satisfying the Life Safety limit state which suggests that urgent planning and needed provisions must be considered.

Table 13. Average exceedance ratio of considered performance levels for far-fault and near-fault earthquakes of the selected template designs.

Template Design ID	Direction	Immediate Occupancy (IO)		Life Safety (LS)		Collapse Prevention (CP)	
		Far Fault	Near Fault	Far Fault	Near Fault	Far Fault	Near Fault
TD_1	X	0.869	0.942	0.440	0.701	0.157	0.421
	Y	0.608	0.837	0.140	0.350	0.020	0.195
TD_2	X	0.878	0.951	0.539	0.792	0.229	0.507
	Y	0.630	0.843	0.163	0.376	0.024	0.209
TD_3	X	0.881	0.944	0.619	0.829	0.403	0.658
	Y	0.586	0.821	0.230	0.491	0.092	0.310
TD_4	X	0.488	0.798	0.248	0.571	0.063	0.389
	Y	0.681	0.908	0.306	0.664	0.111	0.427

This study makes a comparative seismic performance assessment of template RC buildings which represent mid-rise residential building stock constructed per pre-modern codes in Albanian practice. 46 far-fault and 54 near-fault records were selected to evaluate the seismic response of these buildings. Structural models were prepared and simulated, and general properties of the members were determined based on experimental tests. The seismic capacities of each building were estimated by using a structural model which uses fiber element approach using ZEUS NL. The nonlinear dynamic characteristics were represented by ESDOF systems, and their seismic demands were calculated under selected ground motions.

- In buildings designed according to the pre-modern codes, low lateral strength and stiffness are among the main causes of damage observed in the 2019 Durres/Albania earthquakes, as they increase the displacement demands.
- As a result of the non-linear static analysis of the investigated building set, strong beam-weak column behavior is observed in most cases. This control, which was not included in the previous regulations (i.e., KTP-78, 1978), leads to negative collapse mechanisms in existing structures, leading to a decrease in the ductility of the structure. This situation is among the important problems of the existing old reinforced concrete building stock.
- A major problem with template designs is the high displacement demand due to their inadequate lateral load bearing capacity and stiffness. In particular, this weakness is notable in TD-4.
- It is observed that the near-fault records have a tendency of producing higher displacement demands as compared to far-fault ones. This indicates the damage potential of near-fault records due to the various absolute or relative energy potential.
- From the results of the analysis, the near-fault impacts on the response of RC structures were notable on each of the performance limit states.
- Based on the analysis results, decision makers should consider seriously the catastrophic nature of such brittle systems when weighing options for earthquake mitigation since these template designs are of low-quality concrete and designed based on the old guidelines.
- The findings of this study were limited to a small number of building configurations and specific typologies. Further important factors should also be studied to generalize the findings of this study.

Such template designs are good examples of the Albanian building stock as well as many other developing countries. As shown from the results in this paper, they have poor lateral strength for areas which are prone to earthquakes. This happens as a reason of weak material quality, low construction workmanship and especially the lack of modern seismic code requirements at the time these buildings were designed.

High deformation demands are remarkable for buildings to dissipate seismic energy due to this low strength and rigidity. Furthermore, the factors that cause low strength of buildings can influence them to behave in a brittle way. Accordingly, it is unreasonable to expect acceptable earthquake performance from such building stocks.

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