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Nonlinear Dynamic Analysis of Isolated and Fixed-Base Reinforced Concrete Structures

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ABSTRACT

Earthquakes are a major threat to human lives and to the integrity of the infrastructures in seismic regions. Structures are the worst hit with the phenomenal damages due to ground motions resulting from earthquakes. Recent research and studies have led to new techniques to reduce the damages caused by earthquakes on structures and these techniques are applied for innovative structural design. One of the techniques is the base isolation method, which is used to design structures against earthquake damages by using seismic isolators to change the dynamic characteristics of a structure.

In this study, three bay 4- and 8-storey reinforced concrete structures are designed as isolated and fixed-base. Lead-rubber bearing (LRB) is used as an isolation system. Nonlinear behavior of both isolation system and super-structure are considered in the modeling. The behaviors of designed models under dynamic loads are analyzed using Ruaumoko computer software. Erzincan, Marmara and Duzce Earthquakes are chosen as the ground motions. At the end of analysis, period, storey accelerations, inter-storey deformations, base shear forces, plastic hinge locations and weighted damage histories are compared for isolated and fixed-base structures. As a result, the advantages of isolated reinforced concrete structures against fixed-base structures under earthquake are shown.

Keywords: *Lead-rubber bearing; Dynamic analysis; Material nonlinearity; Reinforced concrete structure; Damage analysis*

1. INTRODUCTION

In the last decade, light-weight and high-strength materials have widely been used in the construction of high-rise buildings. These structures generally have flexible and low damping characteristics. Reduction of earthquake effects on buildings has always been an important engineering concern in order to minimize environmental and economic impacts. A popular method of earthquake-resistant design concept is the use of seismic isolation devices which are placed between the superstructure and the substructure to reduce the transmission of seismic force from soil to the structure. The seismic demands on the existing structure are reduced through the isolator's natural action of period elongation, increased damping, and energy dissipation.

One of the goals of seismic isolations is to shift the fundamental frequency of a structure away to a value that is much lower than both its fixed-base frequency and the predominant frequency of earthquakes. This is due to the low horizontal stiffness property of the isolation systems. The other goal of using isolation system is to provide additional energy dissipation, thereby, reducing the transmitted acceleration to the superstructure.

Isolation systems including elastomeric bearings (with and without lead core) have been developed and used practically for aseismic design of buildings during the last 20 years [1-4]. Some studies have investigated different structural control systems for buildings [5-8]. The base-isolated buildings might perform poorly

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because of large isolator displacements due to long period pulses associated in the Near-Fault (NF) motion [9]. The efficiency of providing different lead rubber bearing (LRB) systems for actual RC buildings, in combination with supplemental dampers for reducing the isolator displacements while keeping the inter-storey forces in reasonable ranges, was investigated [10, 11]. The response of this combined isolation action as well as the superstructure behavior seems to be effective for NF ground motions. However, such a complex isolation system leads to undesirable results under moderate or strong far-field earthquake excitations.

The buildings isolated with LRB performed very well during the 1994 Northridge and 1995 Kobe earthquakes confirming the suitability of LRB as a base isolator [12]. The suppression of the dynamic response of tall buildings, supported on elastomeric bearings with both linear and nonlinear behaviors, was studied [13]. The isolated building is modeled as a shear-type structure having one lateral degree of freedom at each storey level. The elastic supports are modeled as an additional degree of freedom having three unknown parameters: mass, stiffness, and damping ratio.

The response of multi-storey buildings and bridges isolated by the LRB is investigated under NF motions [14]. Analytical seismic response of multi-storey buildings isolated by LRB is investigated under NF motions. The superstructure is idealized as a linear shear type flexible building. The force–deformation behavior of the LRB is modeled as bilinear with viscous damping.

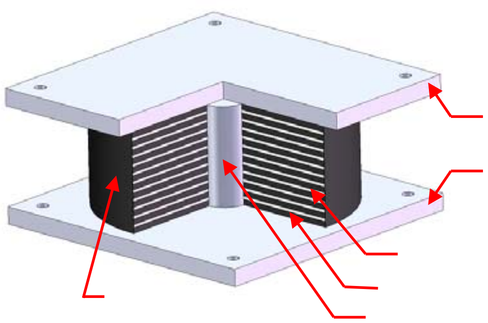
Various LRB isolation systems are systematically compared and discussed for aseismic performances of two actual reinforced concrete buildings [15]. Parametric analysis of the buildings fitted with isolation devices is carried out to choose the appropriate design parameters. The efficiency of providing supplemental viscous damping for reducing the isolator displacements while keeping the substructure forces in reasonable ranges is also investigated.

In this study, three bay 4- and 8-storey reinforced concrete structures are designed as isolated and fixed-base structures. LRB is used as an isolation system. Nonlinear behavior of both isolation system and super-structure are considered in the modeling. The behaviors of designed models under dynamic loads are analyzed using Ruaumoko computer software. Erzincan, Marmara and Duzce Earthquakes are chosen as the ground motions. At the end of analysis, period, storey accelerations, inter-storey deformations, base shear forces, plastic hinge locations and weighted damage histories are compared for isolated and fixed-base structures. As a result, the advantages of isolated reinforced concrete structures are shown against fixed-base structures under earthquake.

2. LEAD RUBBER BEARING

The seismic isolation systems are designed broadly by using elastomers which consist of several elastomer layers sandwiched between and bonded to reinforcing sheets. Elastomeric bearings can be developed using this favorable mechanical property of bonded elastic layers. For increasing the vertical rigidity of an elastomer, lead bars are inserted at its centre. Such systems are called LRB. LRB was invented in 1975 by Dr. Bill Robinson of New Zealand's Department of Scientific and Industrial Research.

LRB is a single compact device reinforced with cylindrical lead cores added to the bearing and this allows LRB to be rigid as well as elastic (Figure 1). These lead bars provide an additional hysteretic damping. When subjected to low lateral loads (such as minor earthquake or wind) the LRB is stiff and remains elastic. The lateral rigidity results from the high elastic stiffness of lead plug. The vertical rigidity (which remains at all load levels) results from the steel–rubber construction of the bearing. At higher load levels, the lead yields and the lateral stiffness of the bearing is significantly reduced to an important degree. This produces the period shift characteristics of base isolation system; hence, the period of the structural system is increased. It is important to note that the transition from low to high load level is smooth, that is, when the lead yields, additional load is carried primarily by the rubber but the load in the lead does not drop. This is a major advantage over other systems which rely on the failure of a wind restraint mechanism and the resulting sudden increase in load carried by bearings [16].



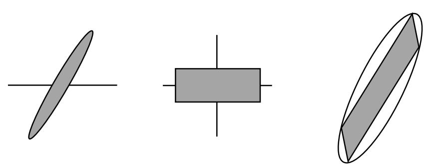


Figure 1. Lead rubber bearing

3. MODELING DETAILS

3.1. Modeling of Superstructure

In this study, three bay 4- and 8-storey structures designed according to Turkish Earthquake Code (2007) [17] are considered as shown in Figure 2. The structures are rectangular in plane (18 m × 12 m) and regular along the height; the inter-storey height is about 3.0 m, while total height is about 12 m for 4-storey and 24 m for 8-storey. Structures are assumed in first-degree seismic zone (high seismically) and Z2-type soil (relatively stiff soil). Concrete material C20 (f_c = 20 MPa) and steel S420 (f_y = 420 MPa) are decided to used in all analysis. Live load 3 KN/m^2 is applied in the storeys, but it is not considered in roof storey. A dead load of 5.25 KN/m^2 is taken into consideration for all storeys. In the beam, besides its own weight, a 5 KN/m^2 extra load is added for the walls.

Reinforced concrete analysis and modeling of the structures are accomplished by using ideCAD structural software [18]. In this study, only B-B axis of 4- and 8-storey structures that have same plan is considered. Dimension of beam is taken as 30 × 50 cm in both structures. In the 4-storey structure, dimensions of axis's outer and inner columns are taken as 45×45 cm and 60×60 cm, respectively. In the 8-storey structure, dimensions of B-B axis's outer and inner columns are considered as 60×60 cm and 90×90 cm, respectively. Dimensions of columns in all storeys are kept constant for both structures.

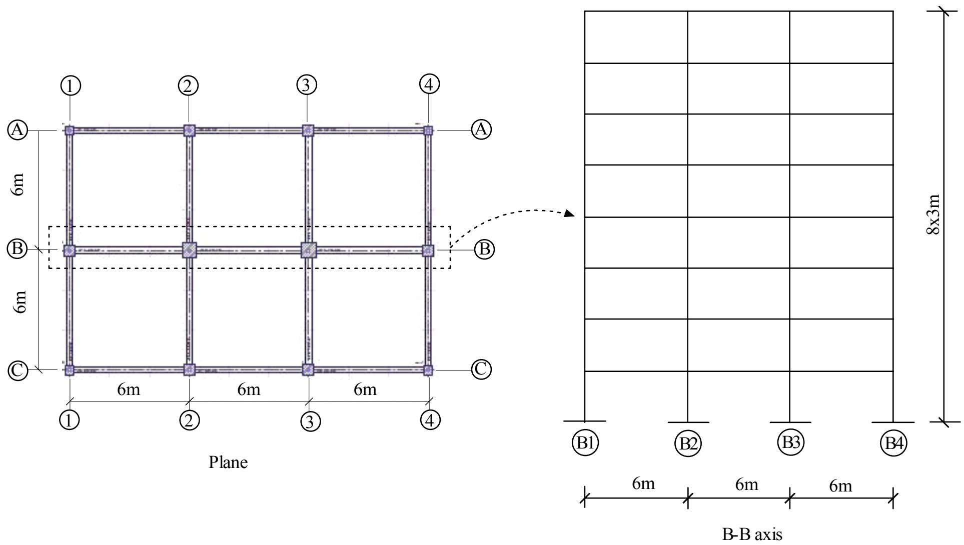


Figure 2. Plane and cross-section of the 8-storey structure

Column and beam steel areas calculated by ideCAD structural software program for the given design parameters are presented in Tables 1-2.

Table 1. Column and beam steel areas for the 4-storey structure

B-B axis	Steel area for beam (cm^2)						Beam dim. (cm×cm)	Column	Long. steel area for column (cm^2)	Column dim. (cm×cm)
Storey	Edge region	Location top	Location bottom	bays	Location top	Location bottom				
4		4.8	2.7		2.26	4.27		B1,B4	20.25	
		5.89	4.27		2.26	4.27		B2,B3	36	
3		10.05	5.72		2.79	4.6		B1,B4	20.25	
	B1,B4	10.87	5.58	B1-B2	2.72	4.23		B2,B3	36	45×45
2	B2,B3	12.98	6.99	B2-B3	3.58	4.58	30×50	B1,B4	20.25	60×60
		14.32	7.85		3.43	4.23		B2,B3	36	
1		14.21	7.23		4.26	5.32		B1,B4	20.25	
		17.05	9.13		3.96	4.61		B2,B3	36	

Table 2. Column and beam steel areas for the 8-storey structure

B-B axis	Steel area for beam(cm^2)						Beam dim. (cm×cm)	Column	Long. steel area for column (cm^2)	Column dim. (cm×cm)
	Edge region	Location		Bays	Location					
Storey		Top	Bottom		Top	Bottom				
8		5.79	2.90		2.26	4.27		B1,B4	36	
		9.91	5.04		2.26	4.27		B2,B3	64	
7		9.29	4.81		3.09	4.52		B1,B4	36	
		10.78	5.76		2.26	4.27		B2,B3	64	
6		11.78	6.08		3.39	4.52		B1,B4	36	
		13.89	7.41		3.09	4.27		B2,B3	64	
5	B1,B4	14.49	7.35	B1-B2	5.08	4.52		B1,B4	36	60×60
	B2,B3	16.82	9.49	B2-B3	4.52	4.27	30×50	B2,B3	64	90×90
4		16.88	8.62		5.08	4.52		B1,B4	36	
		19.10	10.76		5.09	4.27		B2,B3	64	
3		18.67	9.90		6.03	4.52		B1,B4	36	
		20.33	10.76		5.09	4.27		B2,B3	64	
2		18.88	9.90		6.03	4.52		B1,B4	36	
		20.45	10.76		5.09	4.27		B2,B3	64	
1		18.64	9.90		6.03	4.52		B1,B4	36	
		20.62	10.76		5.09	4.27		B2,B3	64	

3.2. Modeling of isolation system

In Turkey, there is no code or standard for seismic isolation for structures. However, Turkish Earthquake Code [17] recommends up-to-date international standards that can be taken into consideration during construction of seismic isolated structures. For this reason, Uniform Building Code (UBC- 97) [19] is considered in the determination of design criteria for base isolation systems. The design parameters are given below.

Seismic zone factor: Zone 4, $Z = 0.40$

Site Soil Profile Category: SD

Seismic Source Type: A type ($M \geq 7$), slip rate $SR \geq 5$ mm/yr

Establishment of the Near-Source Factors: $\Delta > 15$ km, $N_a = 1$ and $N_v = 1$

Design-Basis Earthquake Shaking Intensity: $ZN_v = 0.40$

Maximum Capable Earthquake Response Coefficient: $M_M = 1.25$

Seismic Coefficients: $C_v = 0.64$, $C_a = 0.44$

Effective Damping of the Isolation System: Lead plug-laminated rubber, $\beta_{eff} = 0.15$

Damping Reduction Factor: $B = 1.35$

For isolation design in structures, shear modulus is chosen as $G_A = 0.60$ MPa for outer columns carrying lower loads; however, it is taken as $G_B = 1.2$ MPa for inner columns carrying higher loads. Effective damping of isolators is $\beta = 0.15$, bulk modulus is 2000 MPa and yield stress of lead is 10.5 MPa. The properties of isolators for outer and inner columns in frame systems designed according to UBC 97 are given in Table 3. Lead rubber bearing are designed for an isolation period of $T_b = 2.50$ sec.

Table 3. The properties of isolators for outer and inner columns in frame systems

Isolator properties	Outer Bearing		Inner Bearing	
	4-storey	8-storey	4-storey	8-storey
Bearing Height (mm)	312	312	312	312
Load on bearing (kN)	772	1621	1544	3242
Characteristic strength (kN)	39.7	83	79.4	166
Yield strength (kN)	44.1	92.2	88.2	184.4
Effective stiffness (kN/m)	648	1371	1296	2742
Post-yield stiffness (kN/m)	495	1048	990	2096
Vertical stiffness (kN/m)	530407	1242174	855849	1953362
Bearing diameter(mm)	550	800	550	800
Lead core diameter (mm)	70	100	99	142

Isolators are composed of each 10 mm thick 22 rubber layers and each 2 mm thickness 21 steel layers. Top and bottom of isolators are made up of 25 mm thickness steel plates.

4. ANALYSIS OF STRUCTURES

Ruaumoko [20] software has been used for the performance of nonlinear time-history analysis of the base-isolated and conventional structures designed in this study. The structures have been assumed in the modeling that each storey is a rigid diaphragm and column and beam responses are in elastoplastic behavior. At the time of analysis, three earthquake acceleration records have been used as input excitations in the simulation as mentioned in UBC 97 [19]. The detailed information about these three earthquake records are given below:

(1) Marmara Earthquake E-W component, 17 August 1999, peak acceleration: 373.76 cm/s^2 (Ms = 7.4).

(2) Duzce Earthquake N-S component, 12 November 1999, peak acceleration: 407.68 cm/s^2 (Ms = 7.2).

(3) Erzincan Earthquake E-W component, 13 March 1992, peak acceleration: 470.92 cm/s^2 (Ms = 6.9).

Only 30% of live loads is considered in defining the structure mass in earthquake calculations. Calculated storey mass was distributed evenly to all joints for each storey. In the 4-storey structure, mass of base and top storey is taken as 81.9 ton and that of other storeys is 110.1 ton. In the 8-storey structure, mass of base and top storey is taken as 86.5 ton and that of other storeys as 117.3 ton. Nonlinear behavior of columns and beams were defined by using both moment-curvature and moment-axial force interaction diagrams. Moment-curvature and moment-axial force interaction diagrams were obtained from X-Tract [21] computer program.

First four periods of isolated and fixed-base structures are given in Table 4. Analysis showed that the first period of base-isolated structure is 3.97 times bigger than that of fixed-base for the 4-storey structure and is 2.30 times bigger for the 8-storey structure. Hence, isolation has the effect of lengthening the structural period so it reduces the risk of being in resonance.

Table 4. Periods for isolated and fixed-base structures for various modes

Storey	Support Case	1. mode	2. mode	3. mode	4. mode
		sec.	sec.	sec.	sec.
8	Fixed	1.023	0.296	0.144	0.084
	LRB	2.353	0.540	0.229	0.122
4	Fixed	0.562	0.165	0.084	0.054
	LRB	2.232	0.311	0.132	0.074

Storey displacements of isolated and fixed-base structures based on storey heights were given in Figures. 3-4. As seen in Figures. 3-4, maximum displacements occurred at Duzce Earthquake for 4-storey fixed-base structure and at Marmara Earthquake for 8-storey fixed-base structure but maximum displacements were at Erzincan Earthquake for both base-isolated structures

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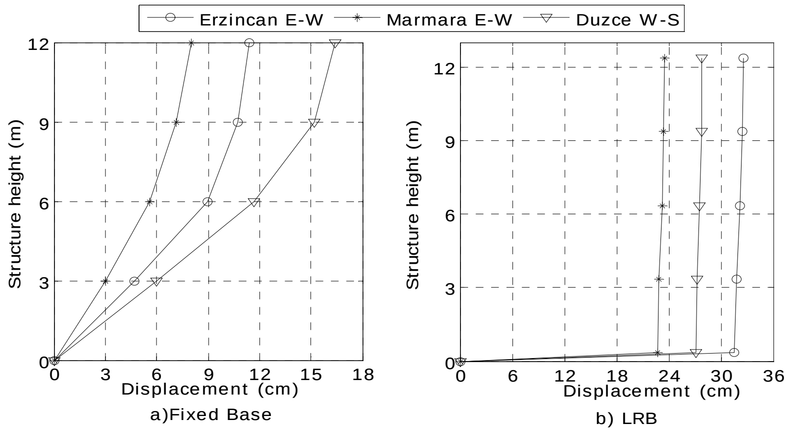


Figure. 3. Variation of displacements with structure height for the 4-storey structure

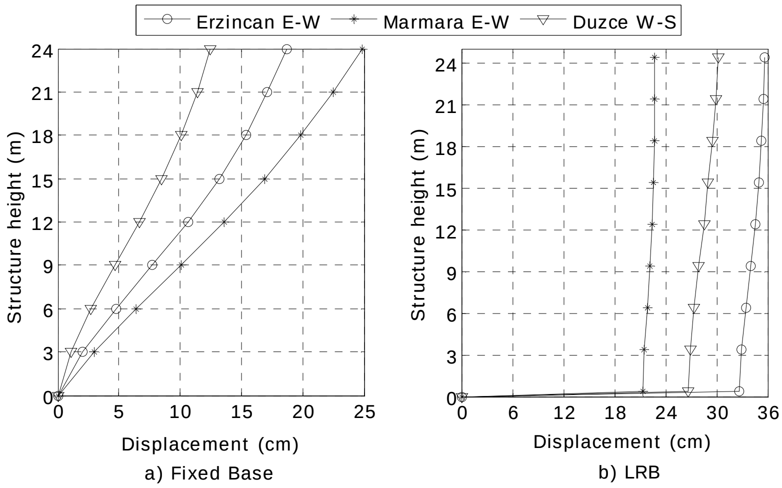


Figure. 4. Variation of displacements with structure height for the 8-storey structure

For base isolated structures, elastic behavior was observed for the investigated 4 and 8 storey models and this is supported with elastic displacement spectrum given in Figure. 5. However, for fixed base structures, inelastic behavior was observed for both 4 and 8-storey models. Due to inelastic behavior, maximum storey displacements were obtained in Duzce Earthquake for 4-storey fixed-base structure and in Marmara Earthquake for 8-storey fixed-base structure.

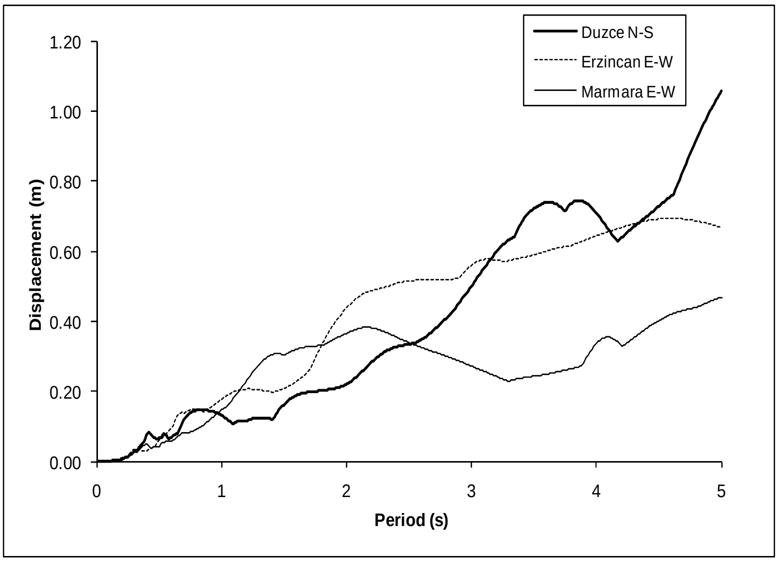


Figure. 5. Elastic displacement spectrum for three earthquakes

Displacements on the base level were increased because of increments in periods of base-isolated structures. However, inter-storey drift of base-isolated structures was lower than that of fixed-base structures. In fixed-base 4-storey structure, the maximum inter-storey drift ratios were 1.57%, 0.99% and 1.99% at the 1st storey for Erzincan, Marmara and Duzce Earthquakes, respectively. In base-isolated structure, the maximum inter-storey drift ratios were 0.12%, 0.11% and 0.11% at the 2nd storey for Erzincan, Marmara and Duzce Earthquakes, respectively. In the 8-storey structure, inter-storey drift ratios were 1.00%, 1.22% and 0.66% at the 3rd storey for fixed-base structures and 0.21%, 0.21% and 0.22% at the 4th storey for base-isolated structures, for Erzincan, Marmara and Duzce Earthquakes, respectively. Inter-storey drifts of isolated and fixed-base structures based on storeys were given in Tables 5-6. This behavior of isolation system provides the minimum damage occurrences in structural members during an earthquake.

Table 5. Inter-storey drifts of base-isolated and fixed-base condition for the 4-storey structure

Storey	Inter-storey drift ratio (%)					
	Erzincan		Marmara		Duzce	
	Fixed	LRB	Fixed	LRB	Fixed	LRB
4	0.32	0.06	0.39	0.05	0.42	0.05
3	0.61	0.10	0.58	0.08	1.17	0.09
2	1.42	0.12	0.87	0.11	1.90	0.11
1	1.57	0.09	0.99	0.08	1.99	0.08

Table 6. Inter-storey drifts of base-isolated and fixed-base condition for the 8-storey structure

Storey	Inter-storey drift ratio (%)					
	Erzincan		Marmara		Duzce	
	Fixed	LRB	Fixed	LRB	Fixed	LRB
8	0.85	0.09	0.95	0.12	0.62	0.10
7	0.88	0.13	1.01	0.16	0.61	0.13
6	0.86	0.17	1.07	0.20	0.57	0.17
5	0.92	0.19	1.15	0.21	0.61	0.20
4	1.00	0.20	1.20	0.21	0.66	0.22
3	1.00	0.21	1.22	0.19	0.66	0.21
2	0.90	0.18	1.16	0.15	0.56	0.18
1	0.69	0.09	0.98	0.07	0.35	0.09

In base-isolated structures, the acceleration values at storeys, are decreased while increasing the periods. The acceleration values of structures having both the base-isolated and the fixed-base are given in Tables 7-8 for 4- and 8-storey structures for Erzincan, Marmara and Duzce Earthquakes, respectively.

Table 7-8 show that transmitted ground accelerations has been decreased at the base isolated system according to fixed base systems. The ratio of top storey accelerations proportioned to ground accelerations for 4-storey structure are obtained in fixed and LRB conditions for Erzincan as 238% and 102%, for Marmara as 209% and 110%, for Duzce as 173% and 84% respectively. These ratios at 8-storey structure for Erzincan as 185% and 104%, for Marmara as 229% and 112%, for Duzce as 294% and 88% in fixed and LRB conditions respectively.

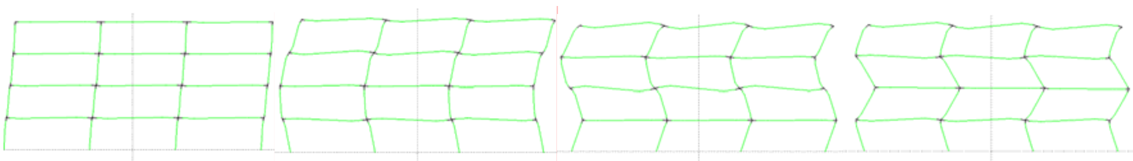
Table 7. Acceleration values of base-isolated and fixed-base for the 4-storey structure

Storey	Absolute Max. Acceleration (m/sn^2)					
	Erzincan		Marmara		Duzce	
	Fixed	LRB	Fixed	LRB	Fixed	LRB
4	11.23	4.83	7.80	4.15	7.06	3.45
3	7.86	4.67	6.08	4.01	5.69	3.63
2	4.44	4.55	4.93	3.96	4.40	3.92
1	3.46	4.32	3.21	4.07	3.22	4.16
Isolation	-	4.13	-	4.12	-	4.33
Ground Motion	4.71	4.71	3.74	3.74	4.08	4.08

Table 8. Acceleration values of base-isolated and fixed-base condition for the 8-storey structure

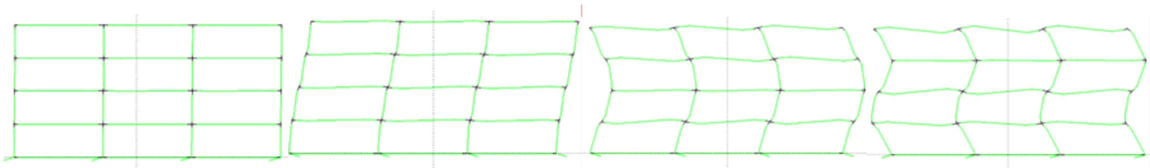
Storey	Absolute Max. Acceleration (m/sn^2)					
	Erzincan		Marmara		Duzce	
	Fixed	LRB	Fixed	LRB	Fixed	LRB
8	8.72	4.94	8.58	4.18	11.99	3.59
7	7.23	4.85	7.33	3.99	8.93	3.57
6	6.95	4.86	5.74	4.09	7.67	3.68
5	7.66	4.87	5.97	4.11	6.11	3.80
4	6.23	4.91	5.63	4.06	4.52	3.91
3	6.25	5	4.28	3.98	4.65	3.98
2	5.29	5.1	3.63	3.88	3.63	3.99
1	3.42	5.15	1.77	3.78	2.15	3.94
Isolation	-	5.16	-	3.72	-	3.90
Ground Motion	4.71	4.71	3.74	3.74	4.08	4.08

Mode shapes of both fixed and base isolated conditions can be seen at Figures. 6-9 for 4 and 8-storey structures. First modal behavior is the most important parameters determining the elastic behavior in structural system under earthquake effects. First modal behavior of the base isolated structures superstructure acts as if a rigid mass. It is also observed that this behavior is within the elastic region in the superstructure and does not reach to the plastic region where permanent damage occurs.



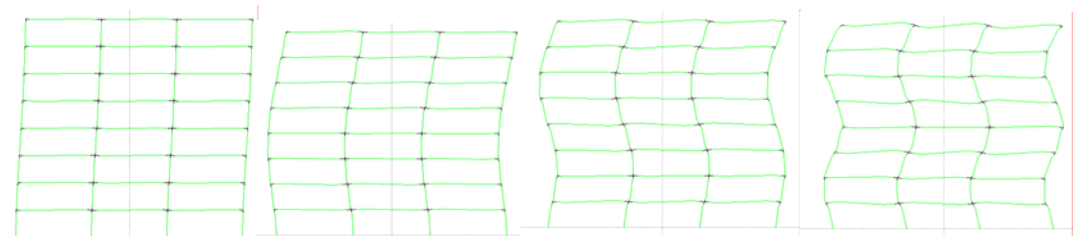
1st Mode 2nd Mode 3rd Mode 4th Mode

Figure 6. Mode shapes for fixed base 4- storey structure



1st Mode 2nd Mode 3rd Mode 4th Mode

Figure 7. Mode shapes for base isolated 4- storey structure



1st Mode 2nd Mode 3rd Mode 4th Mode

Figure. 8. Mode shapes for fixed base 8- storey structure

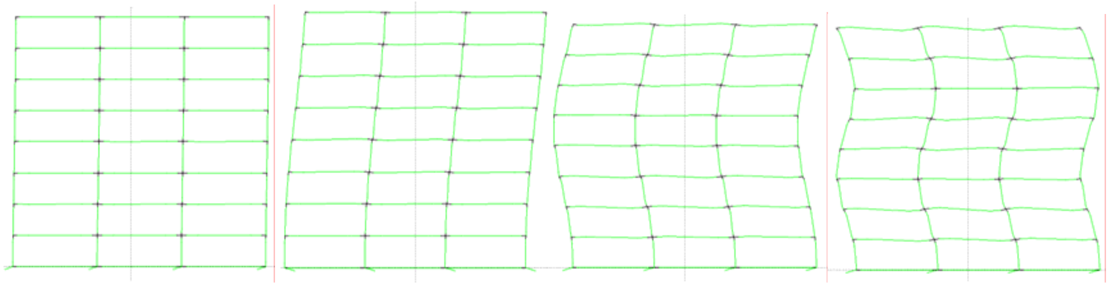


Figure 9. Mode shapes for base isolated 8- storey structure

The storey shear forces acting on the base-isolated systems are decreased as a result of decreasing storey acceleration values. The base shear forces of structures with LRB for Erzincan, Marmara and Duzce Earthquakes are 28% of fixed-base systems at the 4-storey structure; 43%, 38% and 41% at 8-storey structures, respectively. Other shear forces at different storey levels are given in Tables 9-10.

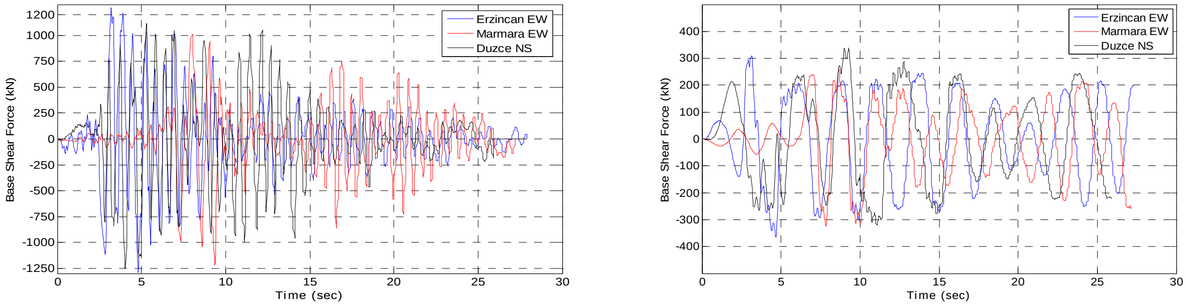
Table 9. Shear forces of base-isolated and fixed-base condition for the 4-storey structure

Storey	Max. Storey Shear Force (kN)					
	Erzincan		Marmara		Duzce	
	Fixed	LRB	Fixed	LRB	Fixed	LRB
4	481.23	84.46	425.56	88.9	371.9	83.6
3	913.12	206.58	826.14	181.7	778.1	196
2	970.8	298.72	932.1	262	976.9	284.1
1	1289.1	364.74	1217.5	323.7	1249.4	338.4

Table 10. Shear forces of base-isolated and fixed-base condition for the 8-storey structure

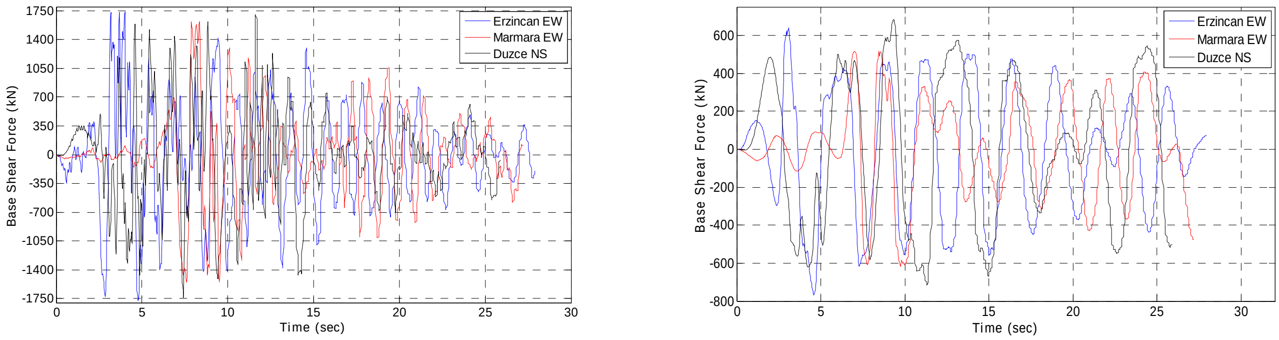
Storey	Max. Storey Shear Force (kN)					
	Erzincan		Marmara		Duzce	
	Fixed	LRB	Fixed	LRB	Fixed	LRB
8	657.1	119.5	423.4	152.9	647	130
7	890.8	289.3	876.6	350.4	1010	310.4
6	1103	415.8	1025.2	485.2	1005.7	448.1
5	1266	501.6	1038.2	559.9	1292.9	563.6
4	1390.5	591.9	1257.2	590.1	1261.5	637.3
3	1494.6	668.4	1402.5	582.6	1304.6	661.7
2	1540	725.4	1501	580	1513.2	695.8
1	1765	766.5	1615.5	614.3	1738.3	715.8

Variation of base shear forces with time for base-isolated and fixed-base structures are given in Figures. 10-11. As seen in Figures. 5-6, base shear forces of base-isolated structures are highly decreased according to base shear forces of fixed-base structures.



a) Fixed-base b) LRB

Figure. 10. Variation of base shear force with time for the 4-storey structure



a) Fixed-base b) LRB

Figure. 11. Variation of base shear force with time for the 8-storey structure

5. DAMAGE ANALYSIS OF STRUCTURES

The damage in reinforced concrete elements will be quantified with the Park and Ang [22] damage index. This index combines the maximum lateral displacement effects with the plastic dissipated energy at one end of the element according to the following expression:

$$D = \frac{\delta_m}{\delta_u} + \frac{\beta}{Q_Y \delta_u} \int dE \qquad (1)$$

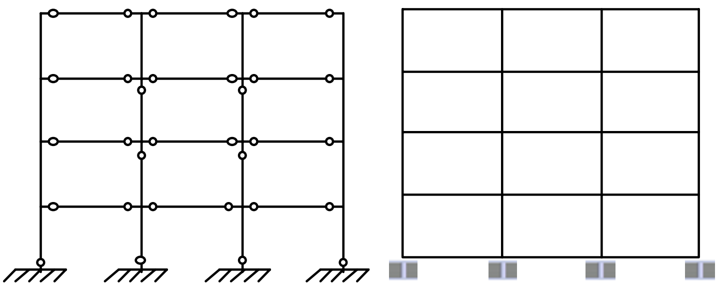
where δ_m is the maximum lateral displacement, δ_u is the ultimate displacement, β is a model constant parameter, $\int dE$ is the hysteretic energy absorbed by the element during the earthquake, Q_y is the yield strength of the element. Damage history of investigated structures for three earthquake records can be seen in Tables 11.

Table 11. Damage history of the 4-8 storey structures

Support case	Fixed-Base			LRB		
Earthquake	Erzincan	Marmara	Duzce	Erzincan	Marmara	Duzce
Maximum member damage index for 4-storey	0.26	0.23	0.41	-	-	-
Weighted damage index for 4-storey	0.095	0.087	0.133	-	-	-
Maximum member damage index for 8-storey	0.23	0.29	0.22	0.053	0.063	0.060
Weighted damage index for 8-storey	0.111	0.136	0.100	0.046	0.046	0.047

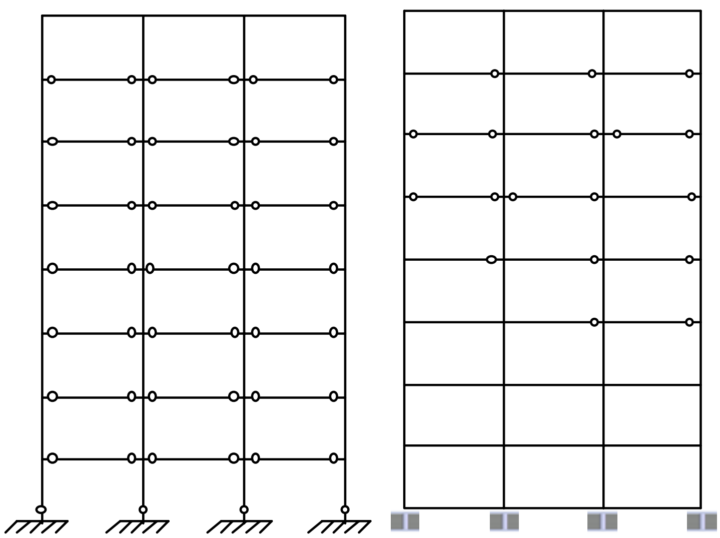
According to three earthquake records, maximum damages were occurred for the 4-storey fixed-base structure at Duzce Earthquake and no damages for the 4-storey base-isolated structure. On the other hand maximum damages were occurred for both of the 8-storey structure at Marmara Earthquake. On the Park–Ang damage index, the weighted damage history of both fixed-base structures falls at minor damage level and of both base-isolated structures is observed as no damage level [22].

If three time-history analyses are performed, then the maximum response of the parameter of interest shall be used for design [19]. Hence Duzce Earthquake for 4-storey and Marmara Earthquake for 8-storey structures are chosen with respect to weighted damage index results. Plastic hinge locations of base-isolated and fixed-base 4- and 8-storey structures are given in Figures. 12-13, respectively. As seen in Figure. 11, the plastic hinges are occurred at all of beams and all of first storey columns, inner columns of second and third storeys of fixed-base 4-storey structure. As given in Figure. 8, only the first storey columns and all of beams except top storey beams have plastic hinges in fixed-base 8-storey structure. In base-isolated structures, plastic hinges are not seen at 4-storey structure but are observed at some beams of 8-storey structure. As shown in Figure. 12, plastic hinge occurrences at beams of fixed-base are more than in the base-isolated 8-storey structure. Plastic hinge occurrences at intermediate storeys in the base-isolated 8-storey structure can be explained as a result of steel area changes. These phenomena can be seen in Tables 1-2.



a) Fixed-base b) LRB

Figure. 12. Plastic hinge locations for the 4-storey structure



2.

a) Fixed-base b) LRB

Figure. 13. Plastic hinges locations for 8-storey structure

6. CONCLUSION

In this study, nonlinear dynamic analyses of 4- and 8-storey reinforced concrete structures as isolated and fixed-base types are performed. In analyses, nonlinear behavior is considered for superstructure and substructure of isolated structures. LRB is used as an isolation system and superiorities of LRB are investigated according to fixed-base systems.

In base-isolated structures large reduction is observed in acceleration values, base shear forces and relative storey displacements with respect to conventional structures. As a result of decreasing relative storey displacements, the accelerations acting on superstructure are damped at base level and the internal forces in superstructures are reduced. On the other hand, the displacements and periods of base-isolated structures are increased comparing with fixed-base structures.

At isolated structures the displacements occurred at the base level are within acceptable values hence the usage of additional dampers is not necessary.

Strong-column and weak-beam rule must be considered at structural design in order to obtain plastic hinge locations at beams. Plastic hinges are observed at columns and beams of fixed-base 4- and 8-storey structures. Plastic hinges occur only at beams of base-isolated 8-storey structure; however, the occurrence of plastic hinges at base-isolated 8-storey structure has not constituted a risk for the reliability of structure. On the other hand, there is no plastic hinge at base-isolated 4-storey structure. Also weighted damage histories are observed for both of fixed-base structures as minor damage level and both of base-isolated structures as no damage level.

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