



Comparative evaluation of RC jacketing, friction-type dampers, and hybrid approaches for the seismic retrofitting of a reinforced concrete building

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Abstract

In this study, a reinforced concrete (RC) building with inadequate seismic performance was designed below the requirements of TBEC 2018 and retrofitted using RC jacketing, friction dampers, and a hybrid approach. Subsequently, seismic performance, damage states, periods, and base shear forces were evaluated through nonlinear static pushover analyses, and their effects on cost were compared with those of the reference structure. Based on this study, it was determined that 50% of the ground and second-floor beams in the reference structure were identified at the Limited Damage (LD) level, while the remaining 50% were classified at the Moderate Damage (MD) level. Among the ground-floor columns, 36% were identified at the Advanced Damage (AD) level and 64% at the Collapse level. Above the second floor, all columns and beams in both the reference and retrofitted structures exhibited damage corresponding to the LD performance level. Accordingly, the reference structure was considered to have reached collapse in accordance with TBEC 2018. Following RC jacketing, 70% of the ground floor beams were identified at the LD level and 30% at the MD level, whereas all second-floor beams remained at the LD level. For the ground- and second-floor columns, 60% were identified at the LD level and 40% at the MD level, corresponding to the Life Safety (LS) or Controlled Damage (CD) performance level. Following friction damper retrofitting, 50% of the ground- and second-floor beams were identified at the LD level and 50% at the MD level. All ground-floor columns were classified at the MD level, whereas the second-floor columns were identified at the LD level, corresponding to the LS performance level. The hybrid retrofit resulted in 40% of the ground-floor columns being identified at the LD level and 60% at the MD level, while 72% of the second-floor columns were identified at the LD level and 28% at the MD level. Finally, 77.5% of the ground-floor beams were identified at the LD level and 22.5% at the MD level. Overall, all strengthening methods significantly improved seismic performance, while RC jacketing and the hybrid system reduced structural periods but increased base shear forces to approximately 1.5 times those of the reference structure. Consequently, foundation moments increased by 358% and 239% in the RC jacketed and hybrid systems, respectively, whereas the friction damper system showed only a 6% increase. These higher foundation demands may increase retrofitting-to-replacement cost ratios for the RC jacketing and hybrid systems. Thus, despite the improved seismic performance, retrofitting cost remains a key factor in strengthening strategy selection.

1. Introduction

Nowadays, various conventional and advanced methods are available to enhance the seismic performance of structures. One of the most widely used strengthening techniques is the application of Carbon Fiber Reinforced Polymer (CFRP). The effects of CFRP systems on structural members such as columns, beams, and beam-column joints have been extensively investigated in the literature [1–7]. Another

widely preferred structural strengthening approach, particularly due to its material accessibility and practical ease of implementation, is the jacketing technique. In this method, structural elements such as columns and beams are retrofitted by encasing them with materials including concrete, reinforced concrete, or steel, thereby enhancing their load-carrying capacity and seismic performance. For instance, Chong et al. [8] employed the steel jacketing method to strengthen a reinforced concrete structure. In another study,

Zhang et al. [9] conducted both experimental and analytical investigations using the Ultra-High-Performance Concrete (UHPC) jacketing technique to improve the seismic performance of reinforced concrete bridge columns. Similarly, Avcil et al. [10] utilized the reinforced concrete jacketing method for the strengthening of reinforced concrete structures. In addition, jacketing techniques for column strengthening can be implemented based on various design standards and methodological approaches [11,12]. Among technologically advanced structural strengthening methods, seismic damper systems exhibit a wide range of applications.

Numerous types of dampers have been investigated in the literature. Among them, viscous dampers [13–15], metallic yielding dampers [16–19], and friction dampers have been widely studied both in terms of the development of new damper models and their effects on structural behavior [20–23]. Both conventional and advanced structural strengthening systems can be applied either individually or in a hybrid configuration in engineering practice. In a study conducted by Suk [24], a friction-based damping system and an isolation system were investigated both independently and in a hybrid configuration. In another study, Ozer and Inel [25] examined the effects of viscous dampers and base isolation systems, both separately and in combination, on reinforced concrete structures with torsional irregularities. Rahnavard et al. [26] combined rubber core and U-shaped metallic dampers to enhance the cyclic performance of reinforced concrete structures. In another study, Suk R. et al. [27] investigated the effects of combining FRP systems and friction dampers on the cyclic behavior of reinforced concrete frames. Similarly, Benavent-Climent et al. [28] proposed a hybrid strengthening approach for reinforced concrete structures by integrating metallic dampers, fiber-reinforced polymer (FRP) laminates, and steel-reinforced polymer (SRP) spikes. Safia A. and Mondal S. [29] proposed a strengthening technique using glass fiber-reinforced polymer (GFRP) and aluminum alloy materials to improve the torsional strength of reinforced concrete beams. Various analytical methods have been developed to evaluate the effects of strengthening systems on the seismic behavior of structures. Among these, time history analysis is widely preferred, particularly in performance-based assessments [30–34]. Although time history analysis is a highly effective method, it is not always practical; therefore, when permitted by national codes, the pushover analysis method is commonly used for structural performance evaluation. Numerous studies employing pushover analysis can be found in the literature [35–38]. Due to architectural constraints, hybrid strengthening strategies are often preferred over a single retrofitting technique.

Numerous studies in the literature have investigated seismic retrofitting strategies based on either individual retrofit techniques or combinations of two or more strengthening methods. In this study, the effects of RC jacketing, friction dampers, and their hybrid combination on the seismic performance and retrofitting cost of reinforced

concrete structures are investigated in detail. In addition to evaluating the resulting changes in structural seismic performance, the differences in the fundamental period, base shear forces, and retrofitting costs among the retrofit configurations are quantified and comparatively assessed. This comparison is particularly important for understanding the potential effects of additional internal forces transferred to the foundation system as a result of the selected retrofit strategy. Within this framework, the seismic performance of an unretrofitted reinforced concrete structure (reference model) was first evaluated using pushover analysis. Subsequently, the structure was retrofitted separately using RC jacketing and friction dampers, and the corresponding seismic performance was assessed. Finally, a hybrid retrofit configuration combining RC jacketing and friction dampers was implemented, and its seismic performance was comparatively evaluated in accordance with the provisions of TBEC 2018. The comparative assessment of these retrofit alternatives provides insight not only into their effectiveness in improving seismic performance but also into their influence on key structural response parameters, particularly the fundamental period and base shear force, which are directly relevant to the demands imposed on the foundation system.

2. Methods and materials

In this study, the structural performance of the building was assessed in accordance with TBEC 2018. Friction damper systems are displacement-dependent devices, and their operating principle is based on Coulomb's law of friction. According to Coulomb's law, the frictional force (F_s) is proportional to the coefficient of friction (μ) and the normal force (N). The Coulomb friction force is defined as shown in Eq. (1) [39].

$$F_s = \mu N \quad (1)$$

According to TBEC 2018, the requirements for reinforced concrete Jacketing method includes some mandatory rules. The concrete jacket thickness must be at least 100 mm. The jacket must extend from the top surface of the floor slab to the bottom surface of the upper floor slab. In addition, TBEC 2018 provides specific requirements for increasing axial compressive strength and for calculating the shear and compressive strengths of the jacketed column [40]. In accordance with TBEC 2018, the pushover analysis procedure illustrated in Fig. 1 was adopted as the governing analysis procedure. As shown in Fig. 1a, in a structure subjected to loading at its roof level, a displacement-dependent reaction force develops, as illustrated in Fig. 1b. The resulting pushover curve is then transformed into a capacity curve, as shown in Fig. 1c. In this transformation, the acceleration parameter (a) is expressed as $V_{base}/\Sigma m$, while the displacement U_{N1} is converted into S_d , yielding the capacity curve.

In the calculation of S_d , Γ represents the modal participation factor, and ϕ_N denotes the amplitude of the fundamental mode shape. On the other hand, the horizontal elastic response spectrum can be obtained from the Turkish Seismic Hazard map, depending on the soil class and the level of seismic hazard. The elastic spectral acceleration (S_{ae}) corresponding to the site and the fundamental period (T) of the structure are presented in Fig. 1d. This spectrum is subsequently transformed into a demand curve, as shown in Fig. 1e.

Finally, as illustrated in Fig. 1f, the capacity and demand curves are intersected to determine the nonlinear spectral displacement (S_{di}), which defines the performance point of the structure. The displacement value required to be applied at the roof level of the structure (U), as a function of S_{di} , ϕ_N , and Γ , can be calculated using Eq. (2).

$$U = S_{di} \phi_N \Gamma C_R \quad (2)$$

In Eq. (2), S_{di} denotes the nonlinear spectral displacement, ϕ_N represents the fundamental mode shape, and Γ is the modal participation factor. C_R is the spectral displacement ratio, which is taken as 1 in accordance with TBEC 2018 when $T_1 > T_A$. The parameter T_B corresponds to the corner period of the elastic horizontal response spectrum. According to the provisions of TBEC 2018, the combined seismic loading used in the pushover analysis is presented in Eq. (3).

$$\begin{aligned} E_d^H &= \pm E_d^x \pm 0.3E_d^y \\ E_d^H &= \pm 0.3E_d^x \pm E_d^y \end{aligned} \quad (3)$$

In Eq. (3), E_d^H represents the directionally combined design seismic load (pushover load), while E_d^x and E_d^y denote the

pushover loads acting in the X and Y directions, respectively. According to TBEC 2018, the vertical seismic effect can be determined as a function of the short-period design spectral acceleration coefficient S_{DS} ($S_{DS}=1.379$), as given in Eq. (4). Considering (Q) as the live load, (S) as the snow load, and E_d^x as the directionally combined design seismic load defined in Eq. (3), the load combinations applied to the structure can be expressed in accordance with TBEC 2018 as presented in Eq. (5). Since the structure is symmetric, only the load combinations in the X-direction were considered, as given in Eq. (5) and Eq. (6).

$$E_d^V = \frac{2}{3} S_{DS} G \quad (4)$$

$$G + 0.3Q + 0.2S + E_d^H + 0.3E_d^V \quad (5)$$

$$\begin{aligned} 1.2758G + 0.3Q + 0.2S + E_d^x + 0.3E_d^y \\ 1.2758G + 0.3Q + 0.2S + E_d^x - 0.3E_d^y \\ 1.2758G + 0.3Q + 0.2S - E_d^x + 0.3E_d^y \\ 1.2758G + 0.3Q + 0.2S - E_d^x - 0.3E_d^y \end{aligned} \quad (6)$$

When structural members are modeled as frame (beam-column) elements, TBEC 2018 adopts the lumped plasticity approach for nonlinear analysis. Based on this approach, the allowable plastic rotation limits occurring in structural members can be calculated using the following [40].

$$\begin{aligned} \theta_p^{CP} &= \frac{2}{3} [(\phi_u - \phi_y) L_p \left(1 - 0.5 \frac{L_p}{L_s}\right) + 4.5 \phi_u d_b] \\ \theta_p^{LS} &= 0.75 \theta_p^{CP} \end{aligned} \quad (7)$$

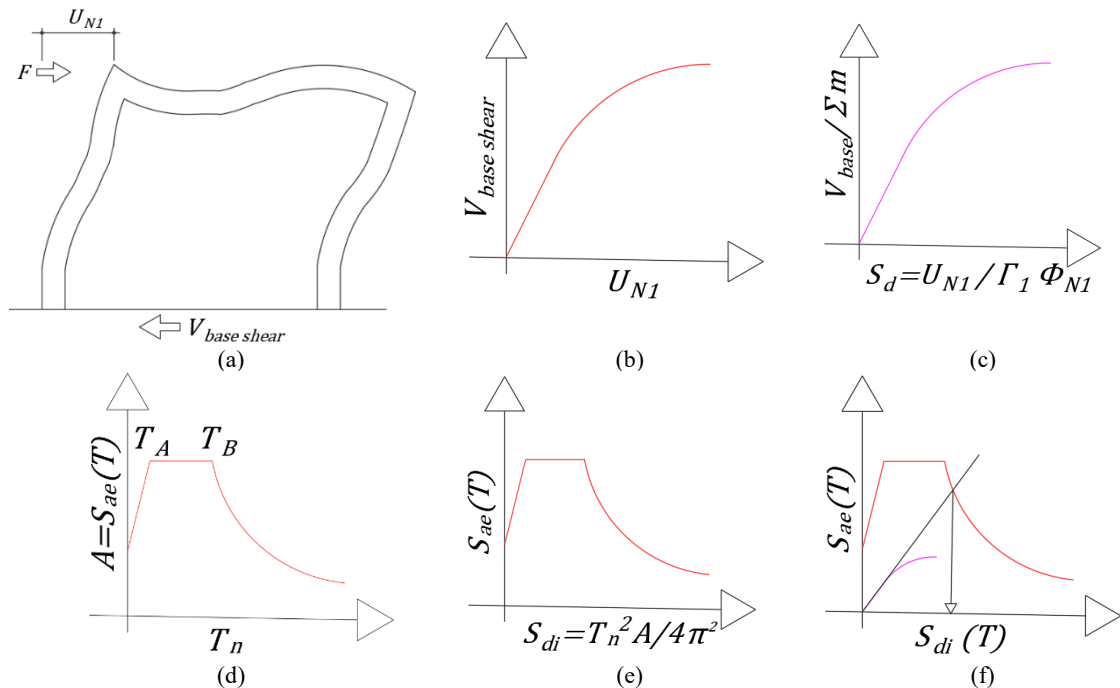


Fig. 1. Pushover analysis procedure (Adapted from [40,43] and redrawn by the author)

where, θ_p^{CP} denotes the allowable plastic rotation limit for the Collapse Prevention (CP) performance level, while θ_p^{LS} represents the allowable plastic rotation limit for the Life Safety (LS) performance level. Furthermore, ϕ_u corresponds to the ultimate curvature (curvature at the onset of collapse), and ϕ_y denotes the yield curvature. L_p is the plastic hinge length, L_s is the shear span, and d_b is the diameter of the longitudinal reinforcement [40]. In addition, for frame elements modeled using the lumped plasticity approach, the effective flexural stiffness of structural members such as columns, beams, or shear walls $(EI)_e$ can be calculated in accordance with TBEC 2018 using

$$(EI)_e = \frac{M_y L_s}{\theta_y} \frac{1}{3} \quad (8)$$

where, M_y denotes the effective yield moment, θ_y represents the yield rotation (rotation at the yield state), and L_s is the shear span. According to TBEC 2018, the yield rotation θ_y can be calculated using Eq. (9). In Eq. (9), θ_y is the Effective yield rotation, d_b is the mean diameter of the reinforcing bars, f_{ye} is the yield strength of the reinforcing steel, f_{ce} is the compressive strength of concrete, and h is the cross-sectional depth. The coefficient η can be taken as 1.0 for columns [40]. A knowledge level of 1.0 was assumed for the structure, and the material strengths for the existing building were taken as $f_{ye} = 420$ MPa and $f_{ce} = 16$ MPa.

$$\theta_y = \frac{\phi_y L_s}{3} + 0.0015\eta \left(1 + 15 \frac{h}{L_s}\right) + \frac{\phi_y d_b f_{ye}}{8\sqrt{f_{ce}}} \quad (9)$$

3. Investigation of the seismic performance of the reinforced concrete structure

In this study, the seismic performance of a four-bay, six-story reinforced concrete structure, symmetric in both principal directions, was evaluated only for the X-direction combinations in accordance with TBEC 2018. Subsequently, the structure was analyzed under four different conditions: the as-built (unretrofitted) case, a retrofit scenario using only reinforced concrete (RC) jacketing, a scenario using only friction dampers, and a final scenario employing a hybrid retrofit approach combining RC jacketing with dampers. The resulting seismic performance responses were then comparatively evaluated in accordance with the provisions of TBEC 2018. In this study, it is assumed that the existing foundation has sufficient capacity to accommodate the additional demands imposed by the strengthening methods.

3.1. Investigation of the seismic performance of the reference structure

The structure was intentionally designed with reduced material properties and modified cross-sectional dimensions

to exhibit inadequate seismic performance, considering that excessively low material strength and reduced cross-sectional properties can lead to premature collapse. In the study, all existing columns have cross-sectional dimensions of 500×500 mm, whereas the beams have cross-sectional dimensions of 250×500 mm.

The structural members were assumed to be constructed using C16-grade concrete and S420-grade reinforcing steel. The floor plan, structural section, and typical reinforcement detailing of the reference building are presented in Fig. 2. The structure was subjected to its self-weight in addition to a uniformly distributed dead load of 3 kN/m² and a live load of 3 kN/m² applied to all floor slabs. Furthermore, the seismic mass was defined in accordance with TBEC 2018 using the mass combination ($G + nQ$), where (G) represents the dead load and (Q) the live load. In this study, the live-load participation factor (n) was taken as 0.3 for residential buildings. In addition, rigid diaphragm constraints were assigned to all floor levels. All beams and columns were modeled as frame elements. To evaluate the damage states of the structural members under seismic loading, a lumped plasticity model was adopted, and plastic hinges were assigned at both ends of all beams and columns. Furthermore, the $P - \Delta$ effect was included in the analysis to account for geometric nonlinearity under gravity loads. The three-dimensional numerical model of the reinforced concrete structure is presented in Fig. 3. In the analyses, vertical loads acting on the structure were also taken into account, and the structure was subjected to roof-level lateral loading. Consequently, the base shear–displacement relationship illustrated in Fig. 1b was obtained. The resulting curve was then transformed into the capacity curve, as shown in Fig. 1c. On the other hand, the target elastic response spectrum used in the analyses (Fig. 1d) was obtained from the Turkish Seismic Hazard Map [41] for the province of İzmir, considering soil class ZC and seismic hazard level DD2.

This target spectrum was subsequently converted into the demand curve, as shown in Fig. 1e. Finally, the performance point of the structure (S_{di}) was determined through the intersection of capacity and demand curves, as illustrated in Fig. 1f. Table 1 summarizes the modal mass participation ratio, story masses (m), ϕ_N , and Γ for the reference structure. Using these parameters, the spectral displacement (S_d) corresponding to each displacement (U_N) was calculated according to Eq. (10), and the resulting $V_{base}/\Sigma m - S_d$ relationship was established to determine the target displacement of the reference structure.

$$S_d = \frac{U_N}{\phi_N \Gamma} \quad (10)$$

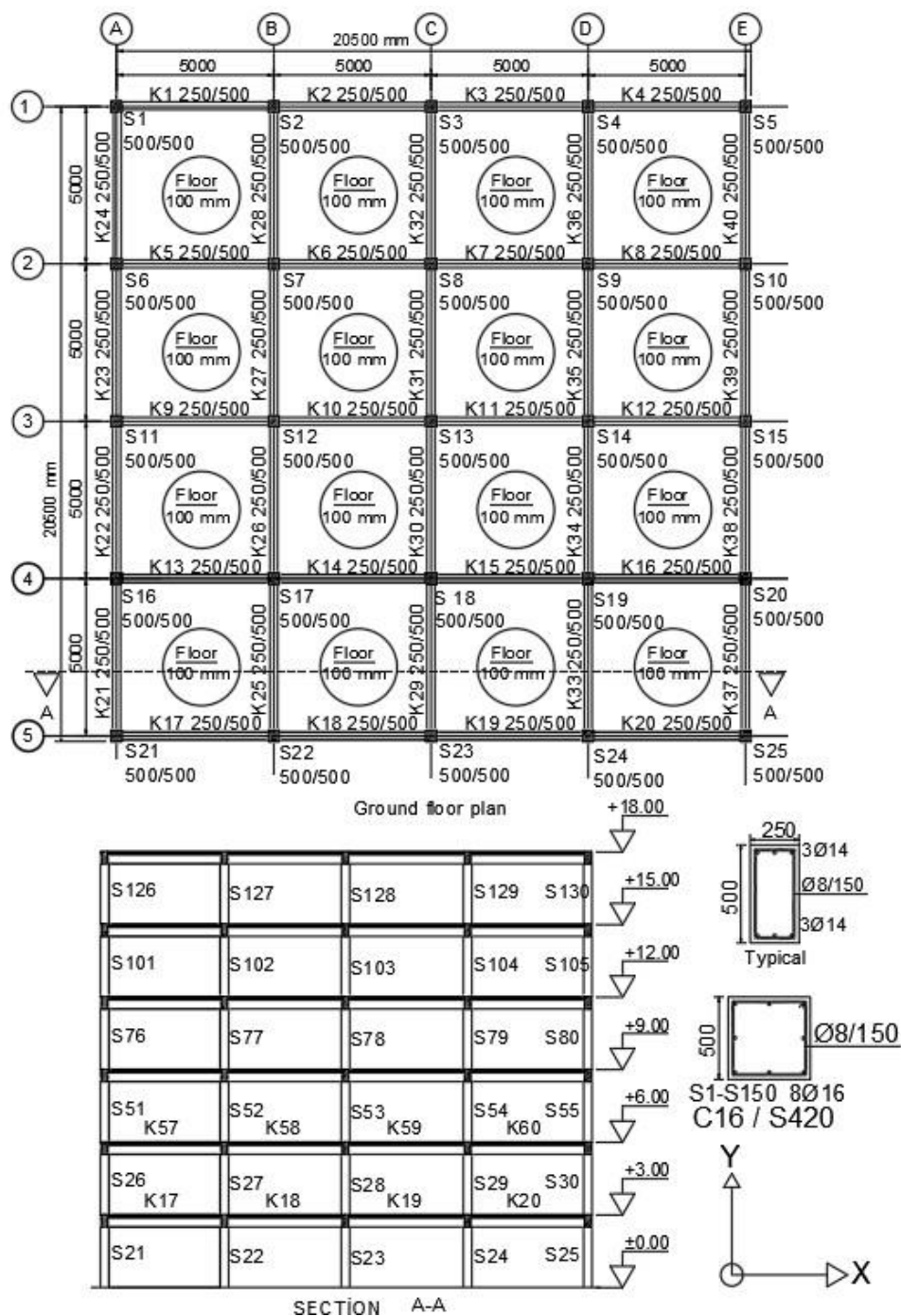


Fig. 2. Typical plan and section of the reference structure

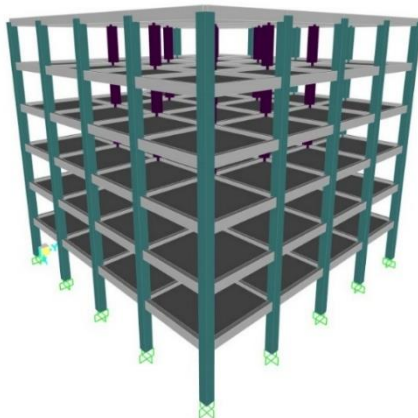


Fig. 3. Analytical model of the reference structure

Table 1. Calculation of the Γ value used in the S_{di} calculation for the reference structure

Story no	Modal mass participation ratio	Mass (kNs ² /m)	Modal amplitude (ϕ_N)	Γ
1	0.81	474.5	0.00390	48.302
2		474.5	0.01010	
3		474.5	0.01610	
4		474.5	0.02010	
5		474.5	0.02470	
6		474.5	0.02670	

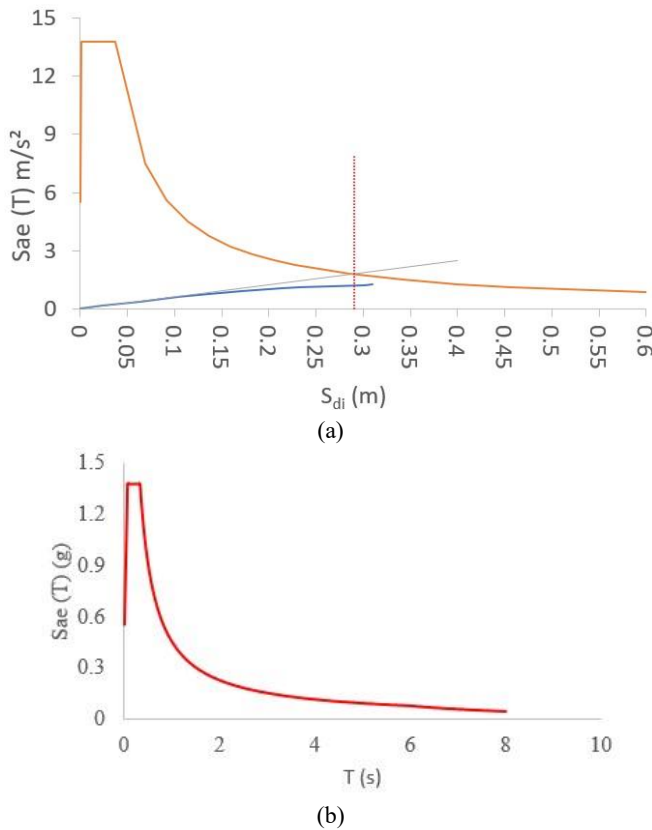


Fig. 4. Reference structure (a) Definition of the performance point (b) Target response spectrum

The resulting curve was then superimposed on the demand spectrum constructed from the target response spectrum using the corresponding spectral acceleration (S_{ae}) and spectral displacement (S_{di}) values. The intersection of these two curves defines the target spectral displacement (S_{di}), as illustrated in Fig. 4(a). The target response spectrum is presented in Fig. 4(b), where $T_A = 0.066$ s and $T_B = 0.326$ s. Finally, the target roof displacement (U) was calculated using Eq. (2) [40]. All pushover analyses were performed using SAP2000 [42].

According to Eq. (2), the structure was subjected to a roof displacement of $U_{roof} = 0.374$ m, and the resulting rotations in the structural members were determined. The value of C_R was taken as 1. In addition, the modal mass participation ratio of the reference structure was calculated as 0.81, and no A1 torsional irregularity was identified due to the structural symmetry. Accordingly, the structure satisfies the

requirements of TBEC 2018 and is suitable for pushover analysis.

Following the pushover analyses, the shear forces, (V_e), developed in the columns and beams of the reference structure were determined. According to TS 500, the shear strength of the cross-section, (V_r), was compared with the design shear force, (V_e), based on Eq. (11) [46].

In Eq. (11), (V_c) is the contribution of concrete to the shear strength, (V_r) is the cracking shear strength, (V_w) is the contribution of shear reinforcement to the shear strength, (f_{ctd}) is the design axial tensile strength of concrete, (A_w) is the web cross-sectional area, (b) is the section width, (A_{sw}) is the total cross-sectional area of shear reinforcement, and (d) is the effective depth. The coefficient (Y) may be taken as 0.07 under compression. Furthermore, (N_d) denotes the axial force, (S) is the spacing between stirrups, and (A_c) is the cross-sectional area of concrete [46]. The parameters adopted for the shear verification of the RC jacketing are as follows: $f_{ywd} = 365$ MPa, $F_{ck} = 30$ MPa.

In addition, the shear strength of the structural members was also evaluated in accordance with TBEC 2018 using Eq. (12). The shear force safety check of the ground floor columns of the reference structure is presented in Table 2, while the shear force safety check of the beams is presented in Table 3. Shear safety was ensured in the columns and beams of the reference structure. Furthermore, the shear force requirements are satisfied for retrofit systems. A strong-column-weak-beam assessment was conducted for the structure, and it was determined that the strong-column-weak-beam requirement specified in TBEC 2018 was satisfied.

$$V_r \geq V_e$$

$$V_r = V_c + V_w$$

$$V_c = 0.8V_{cr}$$

$$V_{cr} = 0.65f_{ctd}b d \left(1 + Y \frac{N_d}{A_c}\right) \quad (11)$$

$$V_w = \frac{A_{sw}}{S} f_{ywd} d$$

$$V_e \leq 0.85 A_w \sqrt{F_{ck}} \quad (12)$$

Table 2. Shear force check of the ground-floor columns in the reference structure

Column	A_{shx}	A_{shy}	d_y (mm)	d_x (mm)	V_e (kN)	N_d (kN)	V_{cr} (kN)	V_c (kN)	V_w (kN)	V_r (kN)	$V_e \leq V_r$
S1	100.48	100.48	470	470	74.67	728.16	250.02	200.0	112.5	312.5	$V_e < V_r$
S2	100.48	100.48	470	470	121.8	1216.3	278.41	222.0	112.5	335.2	$V_e < V_r$
S3	100.48	100.48	470	470	120.9	1183.2	276.49	221.1	112.5	333.7	$V_e < V_r$
S4	100.48	100.48	570	570	122.0	1193.2	368.44	294.7	137.0	431.7	$V_e < V_r$
S5	100.48	100.48	470	470	106.0	1033.8	267.79	214.2	112.5	326.7	$V_e < V_r$
S6	100.48	100.48	470	470	102.1	1193.2	277.06	221.6	112.5	334.1	$V_e < V_r$
S7	100.48	100.48	470	470	116.2	2037.2	326.14	260.9	112.5	373.4	$V_e < V_r$

Table 2. Continued

Column	A_{shx}	A_{shy}	d_y (mm)	d_x (mm)	V_e (kN)	N_d (kN)	V_{cr} (kN)	V_c (kN)	V_w (kN)	V_r (kN)	$V_e \leq V_r$
S8	100.48	100.48	470	470	118.3	1975.4	322.55	258.0	112.5	370.5	$V_e < V_r$
S9	100.48	100.48	470	470	120.6	1999.6	323.96	259.1	112.5	371.7	$V_e < V_r$
S10	100.48	100.48	570	570	117.7	1646.4	394.80	315.8	137.	452.8	$V_e < V_r$
S11	100.48	100.48	470	470	94.47	1179.0	276.24	220.9	112.5	333.5	$V_e < V_r$
S12	100.48	100.48	470	470	117.3	2015.8	324.90	259.9	112.5	372.4	$V_e < V_r$
S13	100.48	100.48	470	470	119.4	1949.4	321.04	256.8	112.5	369.3	$V_e < V_r$
S14	100.48	100.48	470	470	121.3	1975.4	322.55	258.0	112.5	370.5	$V_e < V_r$
S15	100.48	100.48	470	470	118.3	1627.6	302.32	241.8	112.5	354.4	$V_e < V_r$
S16	100.48	100.48	570	570	102.1	1193.2	368.44	294.7	137.0	431.7	$V_e < V_r$
S17	100.48	100.48	470	470	116.0	2037.6	326.17	260.9	112.5	373.4	$V_e < V_r$
S18	100.48	100.48	470	470	118.1	1975.4	322.55	258.0	112.5	370.5	$V_e < V_r$
S19	100.48	100.48	470	470	120.3	1999.6	323.96	259.1	112.5	371.7	$V_e < V_r$
S20	100.48	100.48	470	470	118.7	1646.4	303.42	242.7	112.5	355.2	$V_e < V_r$
S21	100.48	100.48	470	470	75.85	728.16	250.02	200.0	112.5	312.5	$V_e < V_r$
S22	100.48	100.48	570	570	120.8	1211.4	369.50	295.6	137.0	432.6	$V_e < V_r$
S23	100.48	100.48	470	470	120.4	1179.3	276.26	221.0	112.5	333.5	$V_e < V_r$
S24	100.48	100.48	470	470	121.5	1193.2	277.06	221.6	112.5	334.1	$V_e < V_r$
S25	100.48	100.48	470	470	106.5	1030.7	267.62	214.0	112.5	326.6	$V_e < V_r$

Table 3. Shear force check of the ground-floor beams in the reference structure

Beam	A_{shx}	A_{shy}	d_y (mm)	V_e (kN)	N_d (kN)	V_{cr} (kN)	V_c (kN)	V_w (kN)	V_r (kN)	$V_e \leq V_r$
K1	100.48	100.48	460	59.41	1.17	103.91	83.13	112.54	195.66	$V_e < V_r$
K2	100.48	100.48	460	58.51	1.14	103.91	83.12	112.54	195.66	$V_e < V_r$
K3	100.48	100.48	460	58.70	1.14	103.91	83.12	112.54	195.66	$V_e < V_r$
K4	100.48	100.48	460	58.76	1.16	103.91	83.13	112.54	195.66	$V_e < V_r$
K5	100.48	100.48	460	75.92	1.54	103.93	83.14	112.54	195.68	$V_e < V_r$
K6	100.48	100.48	460	72.30	1.50	103.93	83.14	112.54	195.68	$V_e < V_r$
K7	100.48	100.48	460	72.56	1.51	103.93	83.14	112.54	195.68	$V_e < V_r$
K8	100.48	100.48	460	71.55	1.54	103.93	83.14	112.54	195.68	$V_e < V_r$
K9	100.48	100.48	460	75.25	1.56	103.93	83.14	112.54	195.68	$V_e < V_r$
K10	100.48	100.48	460	71.41	1.52	103.93	83.14	112.54	195.68	$V_e < V_r$
K11	100.48	100.48	460	71.69	1.53	103.93	83.14	112.54	195.68	$V_e < V_r$
K12	100.48	100.48	460	71.17	1.56	103.93	83.14	112.54	195.68	$V_e < V_r$
K13	100.48	100.48	460	75.92	1.54	103.93	83.14	112.54	195.68	$V_e < V_r$
K14	100.48	100.48	460	72.29	1.50	103.93	83.14	112.54	195.68	$V_e < V_r$
K15	100.48	100.48	460	72.56	1.51	103.93	83.14	112.54	195.68	$V_e < V_r$
K16	100.48	100.48	460	71.55	1.54	103.93	83.14	112.54	195.68	$V_e < V_r$
K17	100.48	100.48	460	59.44	1.17	103.91	83.13	112.54	195.66	$V_e < V_r$
K18	100.48	100.48	460	58.53	1.14	103.91	83.12	112.54	195.66	$V_e < V_r$
K19	100.48	100.48	460	58.72	1.14	103.91	83.12	112.54	195.66	$V_e < V_r$
K20	100.48	100.48	460	58.78	1.16	103.91	83.13	112.54	195.66	$V_e < V_r$
K21	100.48	100.48	460	46.92	0.16	103.85	83.08	112.54	195.62	$V_e < V_r$
K22	100.48	100.48	460	44.92	0.17	103.85	83.08	112.54	195.62	$V_e < V_r$
K23	100.48	100.48	460	44.92	0.17	103.85	83.08	112.54	195.62	$V_e < V_r$
K24	100.48	100.48	460	46.92	0.16	103.85	83.08	112.54	195.62	$V_e < V_r$
K25	100.48	100.48	460	89.49	0.00	103.84	83.07	112.54	195.61	$V_e < V_r$
K26	100.48	100.48	460	86.76	0.01	103.84	83.07	112.54	195.61	$V_e < V_r$
K27	100.48	100.48	460	87.10	0.01	103.84	83.07	112.54	195.61	$V_e < V_r$
K28	100.48	100.48	460	87.98	0.02	103.84	83.07	112.54	195.61	$V_e < V_r$

Table 3. Continued

Beam	A_{shx}	A_{shy}	d_y (mm)	V_e (kN)	N_d (kN)	V_{cr} (kN)	V_c (kN)	V_w (kN)	V_r (kN)	$V_e \leq V_r$
K29	100.48	100.48	460	84.83	0.01	103.84	83.07	112.54	195.61	$V_e < V_r$
K30	100.48	100.48	460	81.67	0.01	103.84	83.07	112.54	195.61	$V_e < V_r$
K31	100.48	100.48	460	82.11	0.01	103.84	83.07	112.54	195.61	$V_e < V_r$
K32	100.48	100.48	460	83.92	0.01	103.84	83.07	112.54	195.61	$V_e < V_r$
K33	100.48	100.48	460	83.17	0.01	103.84	83.07	112.54	195.61	$V_e < V_r$
K34	100.48	100.48	460	80.32	0.01	103.84	83.07	112.54	195.61	$V_e < V_r$
K35	100.48	100.48	460	80.64	0.00	103.84	83.07	112.54	195.61	$V_e < V_r$
K36	100.48	100.48	460	82.58	0.00	103.84	83.07	112.54	195.61	$V_e < V_r$
K37	100.48	100.48	460	93.09	0.25	103.85	83.08	112.54	195.62	$V_e < V_r$
K38	100.48	100.48	460	88.71	0.26	103.85	83.08	112.54	195.62	$V_e < V_r$
K39	100.48	100.48	460	89.05	0.26	103.85	83.08	112.54	195.62	$V_e < V_r$
K40	100.48	100.48	460	93.65	0.24	103.85	83.08	112.54	195.62	$V_e < V_r$

3.2. Investigation of the seismic performance of the structure retrofitted with RC jacketing

In order to enhance the seismic performance of the structure, a reinforced concrete jacketing system, as shown in Fig. 5, was applied to all columns.

Fig. 5 shows the reinforcement layout of the RC jacketing system, where a jacket thickness of 200 mm was adopted. The longitudinal reinforcement consists of $\varnothing 16$ bars, while $\varnothing 8$ two-leg stirrups are provided at 100 mm spacing within the confinement regions. In addition, cross-ties, as shown in Fig. 5, were incorporated into the design to enhance confinement of the jacketed section. The concrete and reinforcing steel used for the jacketing were C30 and S420, respectively.

The Mander model was adopted to represent the constitutive behavior of the confined concrete. All the aforementioned reinforcement and material parameters were incorporated into the jacketed region of the numerical model. The reinforced concrete jacket was extended continuously through the floor slabs and down to the foundation, thereby ensuring continuity of the strengthening system along the height of the structure. The numerical model of the jacketed structure is presented in Fig. 6. In addition, the axial loads were classified according to interior and exterior columns, with corner columns included in the exterior-column category. The axial loads considered for the ground floor columns are specified in Section 4.2. According to TBEC 2018, the surface of the existing column is roughened prior to jacketing. The transverse reinforcement diameter is not less than $\varnothing 8$, and the stirrup spacing does not exceed half of the minimum cross-sectional dimension or 200 mm. The horizontal spacing between stirrup legs and cross-ties is designed not to exceed 25 times the stirrup diameter. In the confinement regions, the transverse reinforcement detailing is continued in accordance with the requirements of TBEC 2018[40].

According to Eq. (10), the Γ value required for the calculation of S_{di} is presented in Table 4. In addition, the

performance point of the structure after reinforced concrete jacketing is presented in Fig. 7.

Based on Fig. 7 and Eq. (2), the roof displacement to be applied to the structure was calculated as $U_{roof} = 0.283$ m for $C_R = 1$. The modal mass participation ratio was calculated as 0.725, and no A1 torsional irregularity was identified. Therefore, the structure is suitable for pushover analysis.

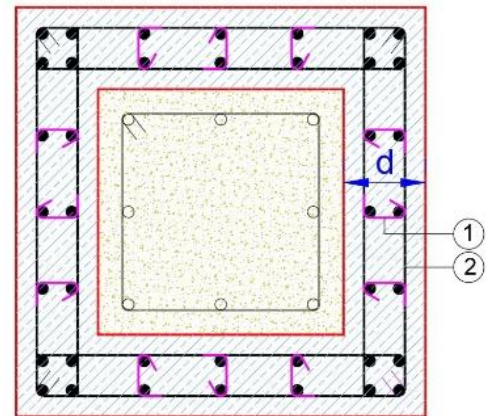


Fig. 5. Reinforced concrete jacketing system

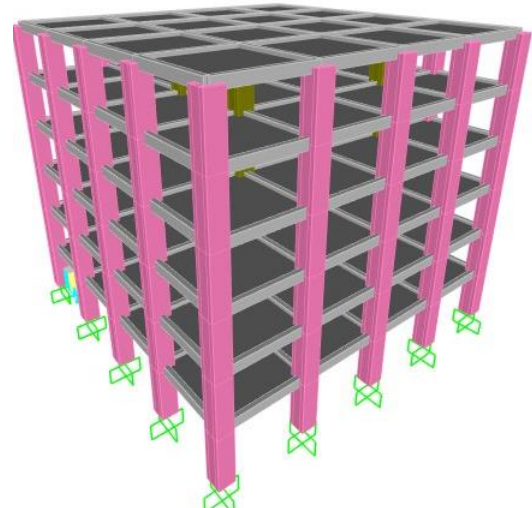


Fig. 6. Structure strengthened with reinforced concrete jacketing

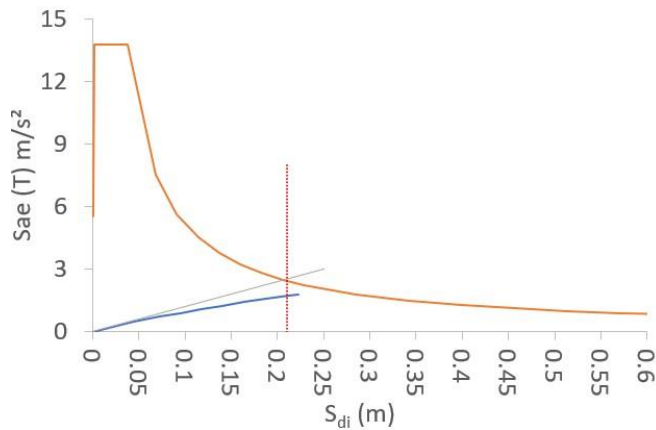


Fig. 7. Performance point evaluation of the RC jacketing structure

Table 4. Calculation of the Γ value used in the S_{di} calculation for the RC jacketed structure

Story no	Modal mass participation ratio	Mass (kNs²/m)	Modal amplitude (ϕ_N)	Γ
1	0.725	581	0.00210	49.458
2		581	0.00690	
3		581	0.01280	
4		581	0.01820	
5		581	0.02330	
6		581	0.02720	

3.3. Investigation of the seismic performance of the structure retrofitted with friction dampers

For the seismic retrofitting of the reference structure, friction dampers with capacities of 50 kN (D50) and 100 kN (D100) were installed in the X and Y directions. In this context, four 100 kN-capacity dampers were placed at the first and second stories, while four 50 kN-capacity dampers were installed at each of the remaining stories. The dampers were integrated into the structural system through an inverted V-type steel bracing configuration, as shown in Fig. 8. In addition, the hysteretic properties of the friction dampers are presented in Table 5. The dampers exhibit an elastic–perfectly plastic hysteretic response and were modeled using the Plastic Wen model. For the initial stiffness value, a yield displacement of $D_y = 3$ mm was assumed, i.e., Initial Stiffness = Yield Strength/ D_y . HEA200 steel sections were employed for the brace members.

For the structure with friction dampers, the Γ value required for the calculation of S_{di} was determined using Eq. (10) and is presented in Table 6. Furthermore, the demand displacement of the structure (S_{di}) was calculated as shown in Fig. 9, yielding a value of $S_{di} = 0.22$ m.

Assuming $C_R = 1$, the roof displacement was determined as $U_{roof} = 0.281$ m in accordance with Eq. (2), and was applied to the structure. In addition, the modal mass participation ratio of the structure retrofitted with friction dampers was calculated as 0.809, and no A1 torsional irregularity was identified.

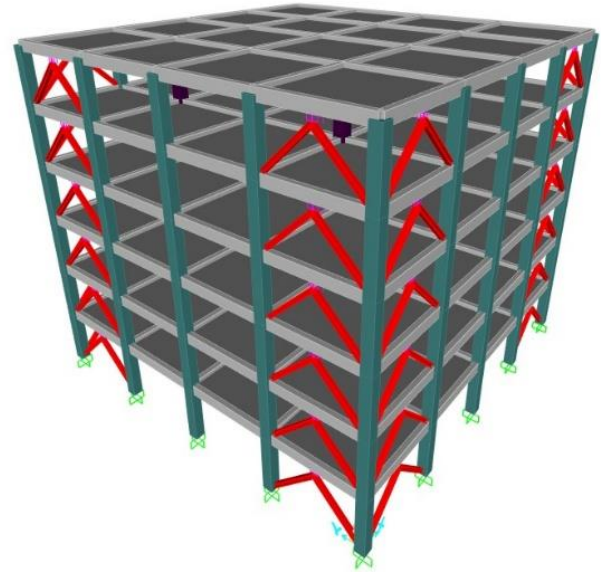


Fig. 8. Installation of friction dampers in the reinforced concrete structure

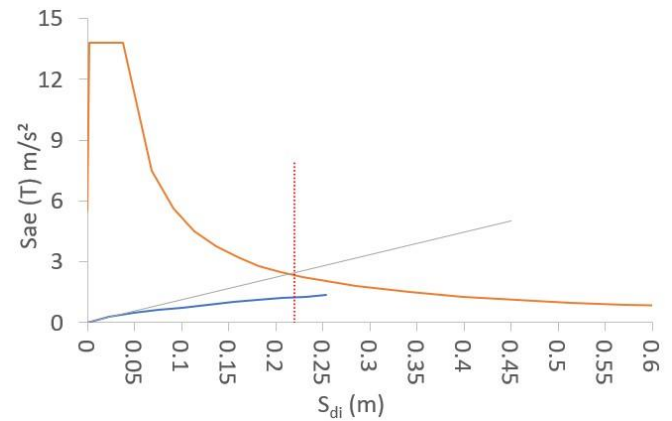


Fig. 9. Performance point evaluation of the structure with dampers

Table 5. Hysteretic properties of the friction dampers

Damper name	Yield strength (kN)	Initial stiffness (kN/m)	Post. Y stiffness ratio	Yielding exp.
D50	50	16670	0.0001	10
D100	100	33333	0.0001	10

Table 6. Calculation of the Γ value used in the S_{di} calculation for the structure with friction dampers

Story no	Modal mass participation ratio	Mass (kNs²/m)	Modal amplitude (ϕ_N)	Γ
1	0.809	476.33	0.00390	47.839
2		476.33	0.01000	
3		476.33	0.01610	
4		476.33	0.02110	
5		476.33	0.02470	
6		476.33	0.02670	

3.4. Investigation of the seismic performance of the hybrid structure

In order to evaluate the seismic performance of the reinforced concrete structure retrofitted with the hybrid system, the analytical model shown in Fig. 10 was developed. The reinforced concrete jacketing described in Section 3.2 was applied only to the interior columns of the structure in this study. Friction dampers were installed along the exterior axes in both the X and Y directions, using the same number and capacity as described in Section 3.3.

According to Eq. (10), the calculation of the Γ value required to determine the S_{di} value of the structure retrofitted with the hybrid system is presented in Table 7. Furthermore, the performance point of the structure retrofitted with the hybrid system was determined as shown in Fig. 11.

As shown in Fig. 11, $S_{d1} = 0.19$ m, and assuming $C_R = 1$, the roof displacement was calculated as $U_{roof} = 0.253$ m. In addition, the modal mass participation ratio of the hybrid-retrofitted structure was calculated as 0.743, and no A1 torsional irregularity was identified.

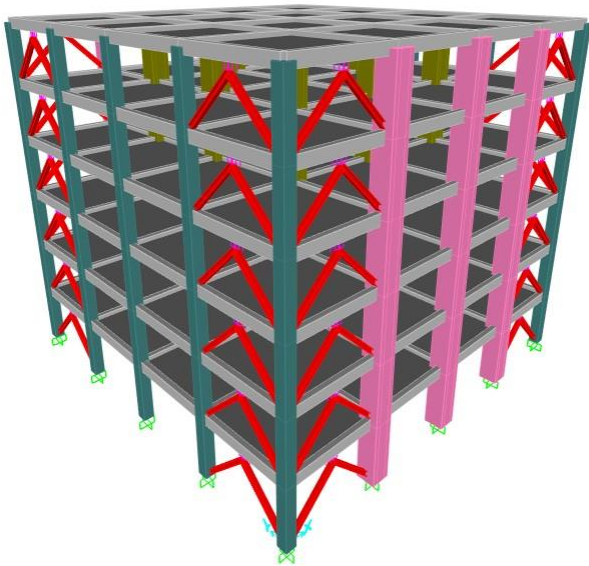


Fig. 10. Hybrid retrofitting model

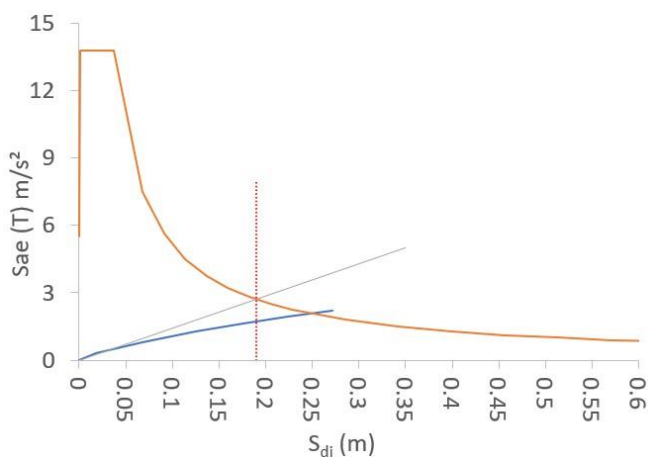


Fig. 11. Performance point evaluation of the hybrid-retrofitted structure

Table 7. Calculation of the Γ value used in the S_{di} calculation for the hybrid-retrofitted structure

Story no	Modal mass participation ratio	Mass (kNs ² /m)	Modal amplitude (ϕ_N)	Γ
1	0.743	543	0.00240	48.592
2		543	0.00750	
3		543	0.01380	
4		543	0.01910	
5		543	0.02380	
6		543	0.02740	

4. Results and discussion

Following the analyses, the damage states of all beams and columns of the structure were evaluated according to TBEC 2018. However, since the ground floor experienced the highest seismic demand, a more detailed assessment of the damage states of the ground-floor beams and columns is presented in this study.

4.1. Damage states of reference structure

In this study, columns with similar axial load values were grouped as interior columns, while edge columns were evaluated in a separate category. Corner columns were also included within the edge-column category.

For the determination of damage states, Eq. (7) of TBEC 2018 was utilized. Within this framework, the damage states of the ground-floor edge and interior columns of the reference structure are presented in Figs. 12 and 13. In addition, the performance states of the ground-floor beams are presented in Fig. 14.

In Figs. 12–14, MD denotes the Moderate Damage level, while C.P. represents the Collapse Prevention level. According to TBEC 2018, the Limited Damage (LD) state corresponds to zero rotation. The region between LD and MD is defined as the Moderate Damage range and is considered to be within the code-prescribed damage limits. The region between MD and C.P. corresponds to the Advanced Damage (AD) state, whereas the region beyond C.P. is defined as the Collapse region. According to TBEC 2018, both the AD and Collapse regions are classified as damaged states.

When the damage states of the columns and beams of the reference structure are evaluated according to story level, Table 8 shows that 36% of the ground floor columns were classified at the AD level, while the remaining 64% were classified in the Collapse state. As shown in Table 9, all first-floor columns were classified at the LD level. The columns on the remaining floors were also classified at the LD level. In addition, Tables 8 and 9 show that, on the ground and first floors, 50% of the beams were classified at the LD level, while the remaining 50% were classified at the MD level. Additionally, the beams on the remaining floors were classified at the LD level.

According to TBEC 2018, for an existing structure to be classified as the LD performance level, no more than 20% of the beams may be within the MD performance level, while all remaining structural members must satisfy the LD performance level. For a structure to be classified as the LS performance level, no more than 35% of the beams may be within the AD performance level, and the contribution of vertical load-carrying members that exceed the AD performance level to the total story shear force in the relevant earthquake loading direction must not exceed 20% of the total contribution of all vertical load-carrying members. For a structure to be classified at the Collapse Prevention performance level, no more than 20% of the beams may be within the Collapse region, while the remaining structural

members may be within the LS, MD, or AD performance levels. If these Collapse Prevention criteria are not satisfied, the structure is classified at the Collapse performance level.

Accordingly, as shown in Table 8, 64% of the columns in the ground floor of the reference structure are within the Collapse region; therefore, the ground floor is classified at the Collapse performance level. According to Table 9, the second floor can be classified at the LS performance level, as no damage was detected in the columns and no damage corresponding to the AD performance level was observed in the beams. Moreover, no damage was observed on the other floors. Considering all these findings, it can be concluded that the structure is at the collapse limit state.

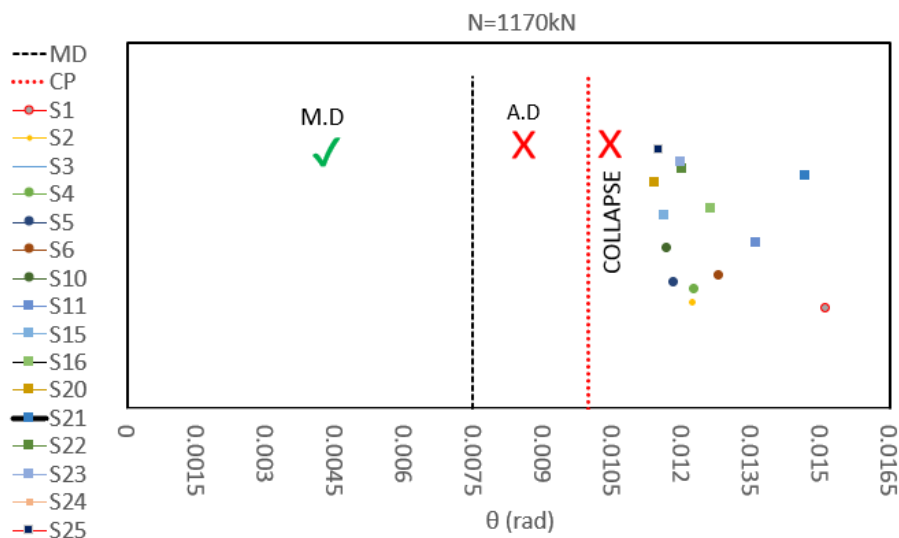


Fig. 12. Damage state of the edge columns at the ground floor of the reference structure

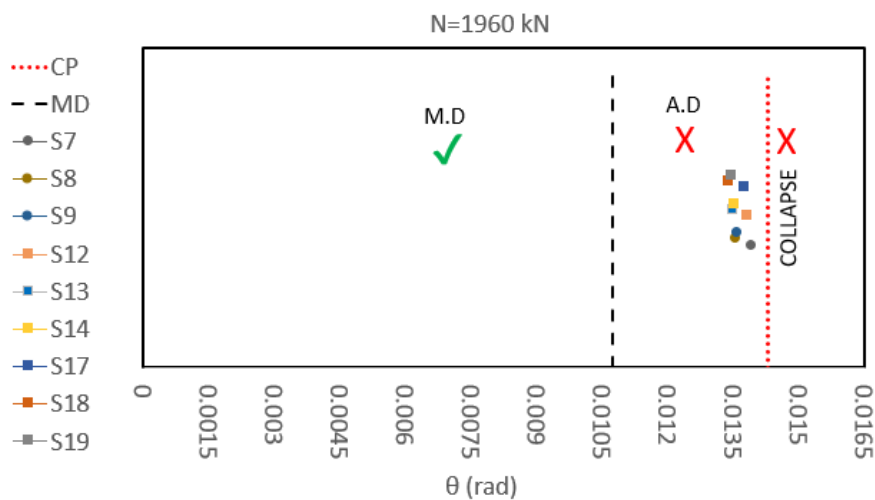


Fig. 13. Damage state of the interior columns at the ground floor of the reference structure

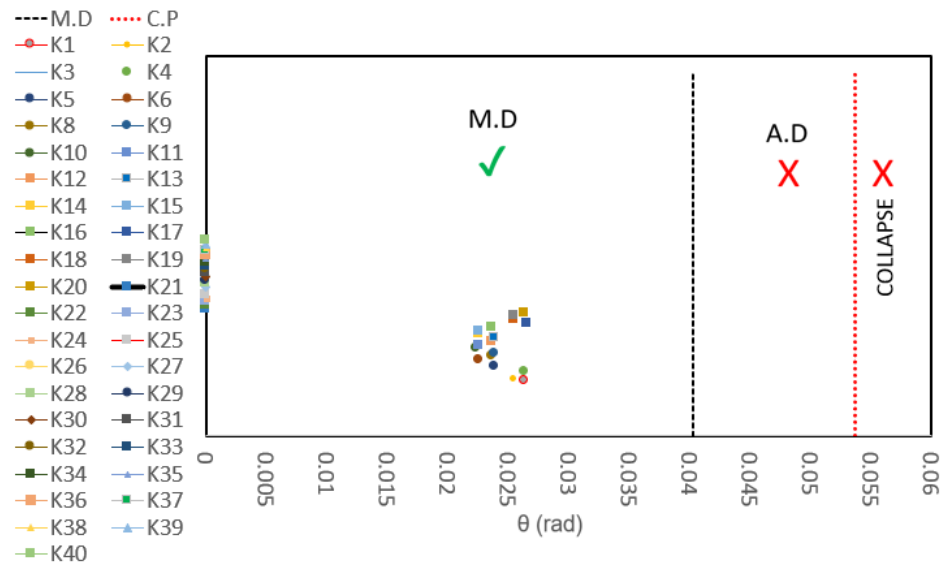


Fig. 14. Damage state of the beams at the ground floor of the reference structure

Table 8. Number of damaged structural members and ground-floor performance level of the reference structure according to TBEC 2018

Damage Type	Beam		Column		Damage status
	Damage number	Damage ratio (%)	Damage number	Damage ratio (%)	
LD	20	50	0	0	Collapse
MD	20	50	0	0	
AD	0	0	9	36	
Collapse	0	0	16	64	
Total	40	100	25	100	

Table 9. Number of damaged structural members and second-floor performance level of the reference structure according to TBEC 2018

Damage Type	Beam		Column		Damage status
	Damage number	Damage ratio (%)	Damage number	Damage ratio (%)	
LD	20	50	25	100	L.S
MD	20	50	0	0	
AD	0	0	0	0	
Collapse	0	0	0	0	
Total	40	100	25	100	

*No damage was observed on the other floors.

4.2. Damage states of the structure retrofitted with RC jacketing

The damage states of the ground-floor edge columns of the structure retrofitted with the RC jacketing system are presented in Fig. 15, while the damage states of the interior columns are shown in Fig. 16. In addition, the damage states of the ground-floor beams are presented in Fig. 17.

As shown in Figs. 15 and 16, the damage states of the reinforced concrete ground-floor columns clearly remain within the LD and MD performance levels. Similarly, Fig. 17 demonstrates that the ground-floor beams also satisfy the LD and MD performance levels. In addition, the damage states of

all ground-floor beams and columns in the structure retrofitted with RC jacketing are presented in Table 10, while those of the second-floor beams and columns are presented in Table 11.

According to Table 10, no structural members at the base floor reached the AD or Collapse performance levels, and the ground floor can therefore be classified at the LS performance level. According to Table 11, no columns or beams on the second and remaining floors reached the AD performance level. Therefore, the seismic performance of the structure retrofitted with RC jacketing was classified at the LS performance level according to TBEC 2018.

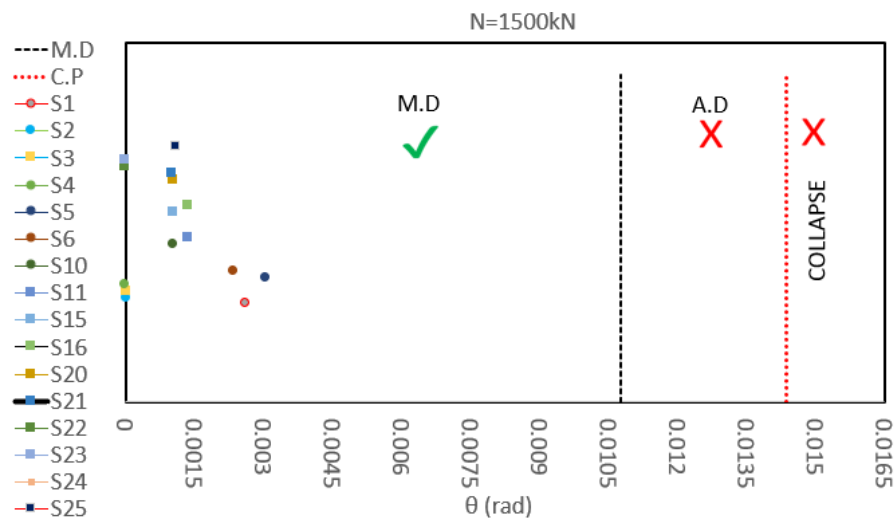


Fig. 15. Damage state of the edge columns at the ground floor, strengthened with RC jacketing

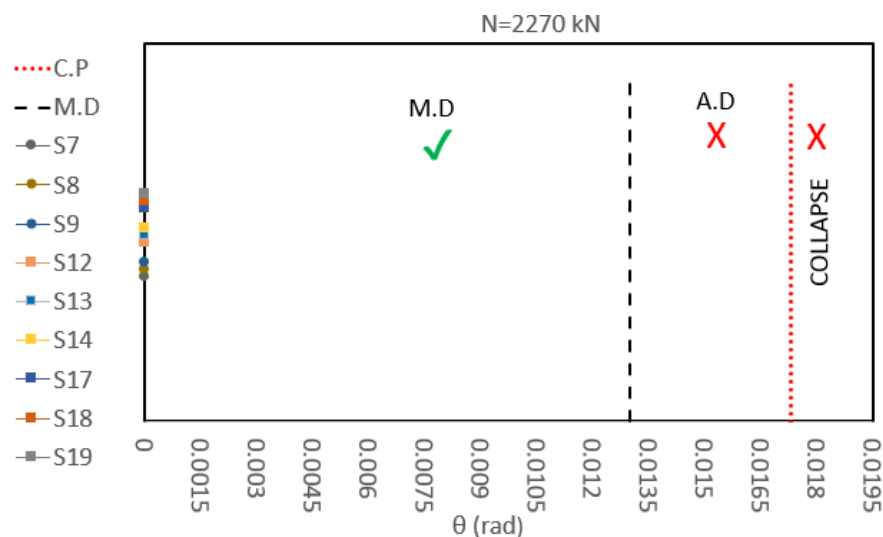


Fig. 16. Damage state of the interior columns at the ground floor, strengthened with RC jacketing

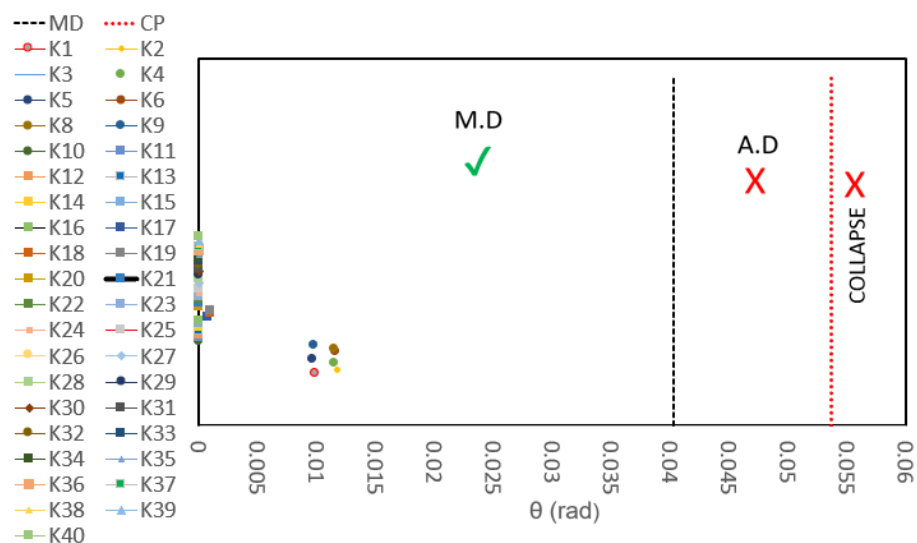


Fig. 17. Damage state of the ground-floor beams strengthened with RC jacketing

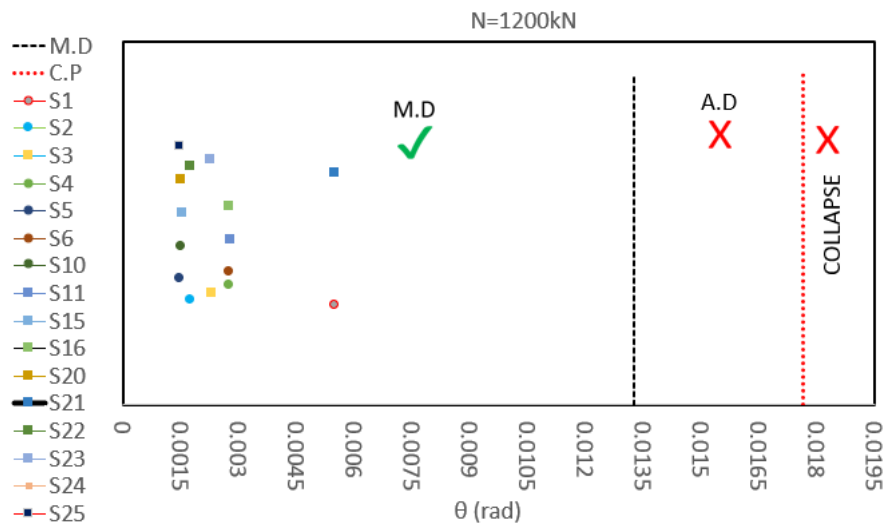
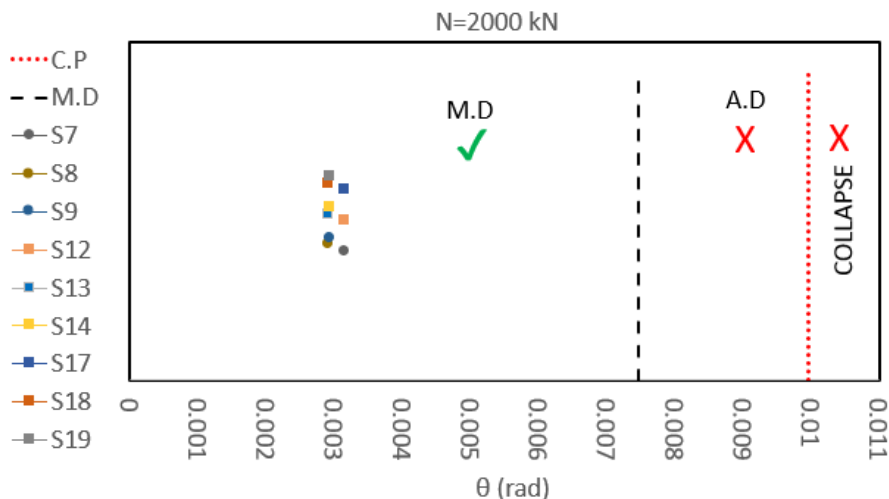
Table 10. Number of damaged structural members and ground-floor performance level of the structure retrofitted with RC Jacketing according to TBEC 2018

Damage Type	Beam		Column		Damage status
	Damage number	Damage ratio (%)	Damage number	Damage ratio (%)	
LD	28	70	15	60	L.S
MD	12	30	10	40	
AD	0	0	0	0	
Collapse	0	0	0	0	
Total	40	100	25	100	

Table 11. Number of damaged structural members and second-floor performance level of the structure retrofitted with RC Jacketing according to TBEC 2018

Damage Type	Beam		Column		Damage status
	Damage number	Damage ratio (%)	Damage number	Damage ratio (%)	
LD	40	100	15	60	L.S
MD	0	0	10	40	
AD	0	0	0	0	
Collapse	0	0	0	0	
Total	40	100	25	100	

*No damage was observed on the other floors

**Fig. 18.** Damage state of the ground-floor edge columns of the structure strengthened with friction dampers**Fig. 19.** Damage state of the ground-floor interior columns of the structure retrofitted with friction dampers

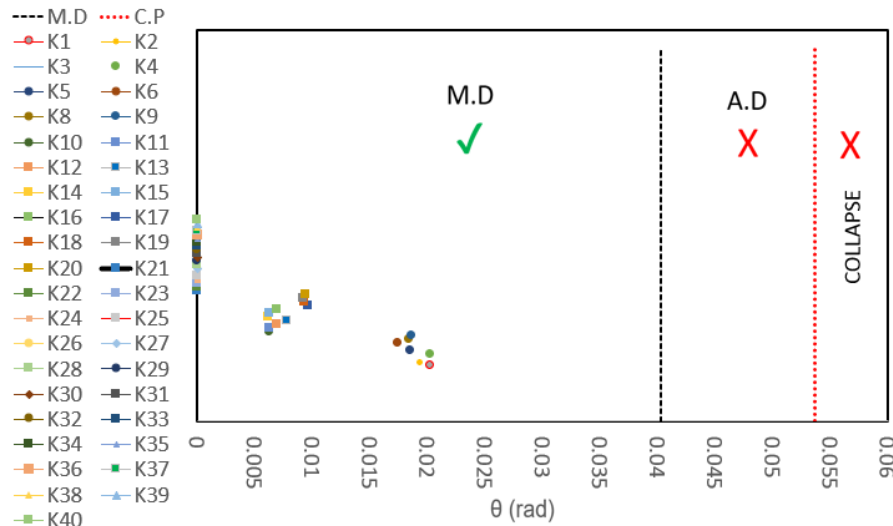


Fig. 20. Damage state of the ground-floor beams of the structure retrofitted with friction dampers

Table 12. Number of damaged structural members and ground-floor performance level of the structure retrofitted with friction dampers according to TBEC 2018

Damage Type	Beam		Column		Damage status
	Damage number	Damage ratio (%)	Damage number	Damage ratio (%)	
LD	20	50	0	0	L.S
MD	20	50	25	100	
AD	0	0	0	0	
Collapse	0	0	0	0	
Total	40	100	25	100	

Table 13. Number of damaged structural members and second-floor performance level of the structure retrofitted with friction dampers according to TBEC 2018

Damage Type	Beam		Column		Damage status
	Damage number	Damage ratio (%)	Damage number	Damage ratio (%)	
LD	20	50	25	100	L.S
MD	20	50	0	0	
AD	0	0	0	0	
Collapse	0	0	0	0	
Total	40	100	40	100	

*No damage was observed on the other floors

4.3. Damage states of the reinforced concrete structure retrofitted with friction dampers

When the structure is retrofitted solely with friction dampers, the damage states of the edge and interior columns are presented in Figs. 18 and 19, respectively. In addition, the performance states of the beams are presented in Fig. 20.

When Figs. 18 and 19 are evaluated together, it can be observed that, in the present study, approximately 100% of the ground-floor columns of the structure retrofitted solely with friction dampers satisfy the damage limit state requirements specified in TBEC 2018. Furthermore, as shown in Fig. 20, all beams remain within the LD and MD performance levels. In addition, the damage states of all ground floor beams and columns in the structure are presented

in Table 12, while those of the second-floor beams and columns are presented in Table 13.

According to Tables 12 and 13, and in accordance with TBEC 2018, the overall structural performance cannot be classified at the LD performance level because some columns remain at the MD performance level. Furthermore, since fewer than 35% of the beams achieve the AD performance level and none of the columns reaches the AD performance level, for this reason, the overall structural performance can be classified at the LS performance level.

4.4. Damage states of the R.C structure retrofitted with the hybrid system

When the structure was retrofitted using both friction dampers and RC jacketing, the resulting damage states of the ground-floor columns are presented in Fig. 21 and Fig. 22.

According to Figs. 21 and 22, all columns retrofitted using the hybrid retrofitting system satisfy the damage limit state requirements specified in TBEC 2018. In addition, the

damage states of the ground- and second-floor structural members in the hybrid-retrofitted structure are presented in Tables 14 and 15, respectively.

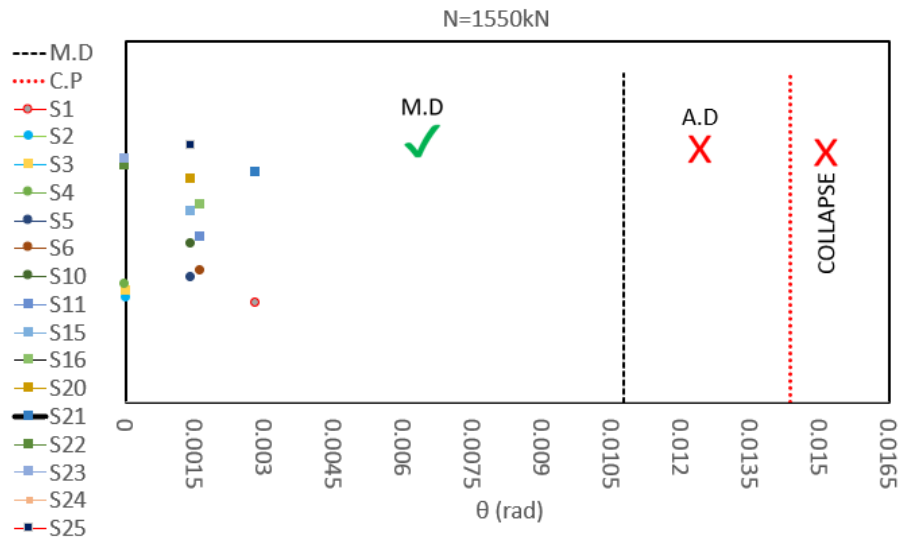


Fig. 21. Damage state of the ground-floor edge columns of the structure strengthened with the hybrid system

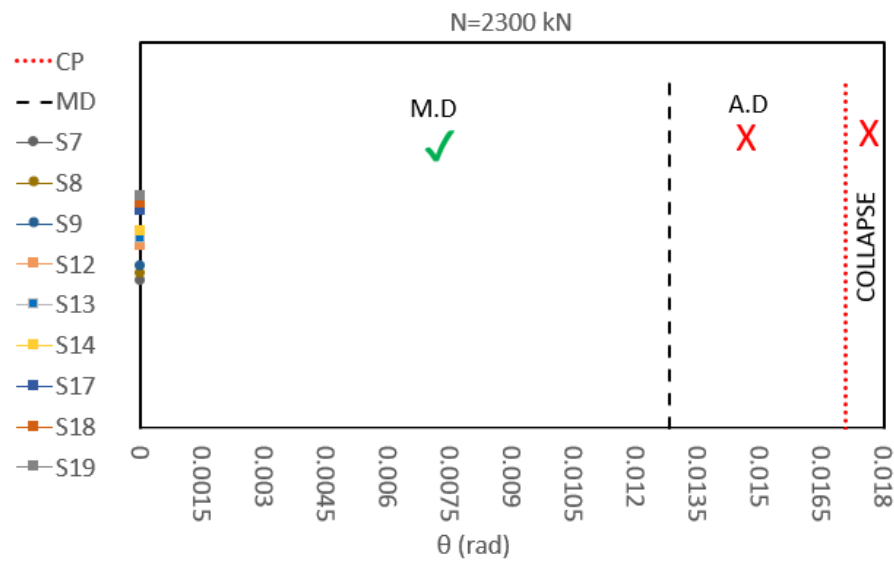


Fig. 22. Damage state of the ground-floor interior columns of the structure strengthened with the hybrid system

Table 14. Number of damaged structural members and ground-floor performance level of the hybrid-retrofitted structure according to TBEC 2018

Damage Type	Beam		Column		Damage status
	Damage number	Damage ratio (%)	Damage number	Damage ratio (%)	
LD	31	77.5	10	40	L.S
MD	9	22.5	15	60	
AD	0	0	0	0	
Collapse	0	0	0	0	
Total	40	100	25	100	

Table 15. Number of damaged structural members and second-floor performance level of the hybrid-retrofitted structure according to TBEC 2018

Damage Type	Beam		Column		Damage sStatus
	Damage number	Damage ratio (%)	Damage number	Damage ratio (%)	
LD	40	100	18	72	L.S
MD	0	0	7	28	
AD	0	0	0	0	
Collapse	0	0	0	0	
Total	40	100	25	100	

*No damage was observed on the other floors

4.5. Comparison of pushover analysis outputs

The base shear force, roof displacement (U_{roof}), fundamental period, and other parameters obtained from the pushover analyses of the reference, RC jacketed, friction-damper-retrofitted, and hybrid structures are presented in Table 16. In addition, the base shear force–displacement relationships for each structural system are shown in Fig. 23.

According to Table 16, the fundamental periods of the reference structure and the structure equipped with friction dampers are observed to be very close to each other. When the fundamental periods of the retrofitted structures are compared with those of the reference structure, the period of the friction damper-equipped structure is found to be 0.087 s lower than that of the reference structure. This reduction is 0.42 s for the RC jacketed structure and 0.504 s for the hybrid structure. This indicates that retrofit approaches incorporating RC jacketing increase the stiffness of the reference structure to a greater extent than the friction-damper-based system. Furthermore, when the base shear forces of the retrofitted structures are compared with those of the reference structure, the base shear force of the friction damper-equipped structure is found to be only 1.029 times that of the reference structure. This ratio is 1.52 for the RC jacketed structure and 1.498 for the hybrid system. Since the increase in base shear forces may induce additional stresses in the foundation members, these findings indicate that the effects of superstructure retrofit methods on the foundation should also be taken into consideration.

Furthermore, when Table 16 and Fig. 23 are considered together, a clear relationship can be observed between the base shear force and the fundamental period. As the fundamental period decreases, the displacement-dependent base shear force increases. For example, the fundamental period of the Reference structure is 2.244 s, with a base shear force of 2811.76 kN. When the structure is retrofitted using the hybrid system, the fundamental period decreases to 1.74 s, while the base shear force increases to approximately 4212 kN. This directly affects the Uroof displacement. The primary reason for this behavior is the additional stiffness introduced into the structure by the retrofit elements.

Furthermore, following the analyses, the maximum axial force and moment reaction values that may develop at the

foundation of the structure at the pushover performance level are presented in Tables 17 and 18, respectively.

According to Table 17, it is observed that the axial force does not increase in the structure with friction dampers, whereas it increases by 13.54% in the structure retrofitted with RC jacketing and by 13.59% in the hybrid structure. Furthermore, according to Table 18, the increase in moment in the structure with friction dampers is approximately 6% relative to the reference structure, whereas this increase reaches 358% in the RC jacketed structure and 239% in the hybrid structure.

When Tables 17 and 18 are evaluated together, it can be concluded that the foundations of the structures retrofitted with RC jacketing and hybrid systems require careful assessment of their structural adequacy and significant strengthening. In contrast, the friction damper-equipped structure, exhibiting results close to those of the reference structure, is expected to require substantially less foundation strengthening.

From a cost perspective, although the literature provides limited information on the proportion of friction dampers in the total construction cost of a building, several studies have highlighted their potential economic benefits. Yıldırım et al. [48] emphasized that conventional design approaches do not always yield the most cost-effective solutions and that supplementary systems, such as dampers, may therefore be considered viable alternatives. Similarly, Pall and Pall [45] reported that the use of friction dampers resulted in overall building cost savings of 1.25% in one building and 1.50% in another. For comparison, Ünal G. et al. [49] reported that the RC jacketing system was applied to approximately 75% of the ground-floor columns and that the retrofitting cost, including foundation strengthening, corresponded to approximately 45% of the total initial construction cost of the building. It was also reported that FRP retrofitting, when applied to the entire structure, corresponded to 32% of the total building cost, whereas shear-wall retrofitting, including foundation strengthening, corresponded to approximately 31% of the total building cost. On the other hand, Yıldırım S. et al. [50] reported that the cost of retrofitting a precast structure with friction dampers corresponded to approximately 27% of the replacement cost. It was stated that architectural repairs were

included in this figure. They also discussed the short construction period.

Considering the increase ratios of the foundation internal forces relative to the reference structure presented in Tables 17 and 18, it can be inferred that the RC jacketing and hybrid retrofitting systems may require significant foundation strengthening, whereas the friction-damper-equipped structure may require only partial foundation strengthening. Furthermore, considering the retrofitting cost-to-replacement-cost ratios considering the foundation

strengthening requirements and the retrofitting cost-to-replacement-cost ratios, the friction damper system is expected to have a lower retrofitting cost relative to the replacement cost than both the RC jacketing system and the jacketing-dominant hybrid system.

For the most cost-effective retrofit system, iterative analyses should be conducted to determine the optimum capacity, number, and configuration of the retrofit elements while satisfying the performance limits specified in TBEC 2018.

Table 16. Pushover parameters and fundamental periods according to the structural system

Structure Type	T_1 (s)	Γ	$\Sigma\varphi_N$	$S_{ae}(T_1)$ m/s ²	S_{d1} (m)	U_{roof} (m)	Base Shear (kN)	Base Shear Ratio
Reference	2.244	48.302	0.0267	1.9	0.29	0.374	2811.76	1
With friction damper	2.157	47.839	0.0267	2.4	0.22	0.281	2892.79	1.029
With R.C jacketing	1.752	49.458	0.0272	2.5	0.21	0.283	4269.95	1.52
Hybrid	1.740	48.592	0.0274	2.7	0.19	0.253	4212	1.498

Table 17. Maximum axial force values of the reference and retrofitted structures and their increase ratios (%) relative to the reference structure

	Reference	With friction damper	With RC jacketing	Hybrid
Axial Load (max) (kN)	2038	2033	2314	2315
$F_{\text{Retrofitted}}/F_{\text{reference}}$	1	0.99	1.1354	1.1359
Increase ratio (%)	0	1	13.54	13.59

Table 18. Maximum moment values of the reference and retrofitted structures and their increase ratios (%) relative to the reference structure

	Reference	With friction damper	With RC jacketing	Hybrid
Moment (max) (kN-m)	330	309	1511	1119
$M_{\text{Retrofitted}}/M_{\text{Reference}}$	1	0.94	4.58	3.39
Increase ratio (%)	0	6	358	239

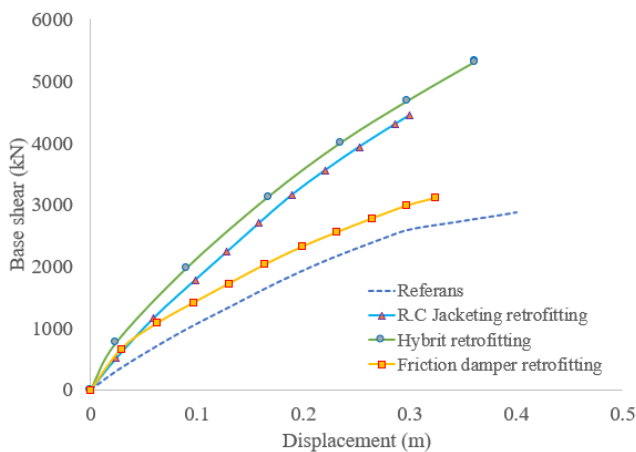


Fig. 23. Comparison of base shear–displacement curves for the structures

5. Validation of pushover analyses using time-history analyses

Following the pushover analysis of the structures equipped with friction dampers, the force–displacement hysteresis loops of the dampers cannot be obtained under a single monotonic loading or unloading condition. In this method, the

damper stiffness is incorporated into the structural system, allowing the damper response to be investigated in greater detail. Therefore, time-history analyses were performed for both the reference and friction damper-equipped structures to verify the results obtained from the single-mode pushover analysis method.

For this purpose, two ground-motion records obtained from the PEER Ground Motion Database were selected: RSN 953-1994, Northridge-01, recorded at the Beverly Hills station, and RSN 1111-1995, Kobe, Japan, recorded at the Nishi-Akashi station [44]. The selected records were scaled to match the target spectrum shown in Fig. 24(b), as presented in Fig. 24. Fig. 24(a) presents the unscaled records, whereas Fig. 24(b) shows the corresponding scaled records.

5.1. Reference structure time-history analysis

Since all ground floor columns of the reference structure were classified as collapsed or severely damaged according to TBEC 2018 in the pushover analysis, the damage states of these columns were also verified using the time-history analysis method. Following the time-history analysis, the damage states observed in the columns are presented in Fig. 25.

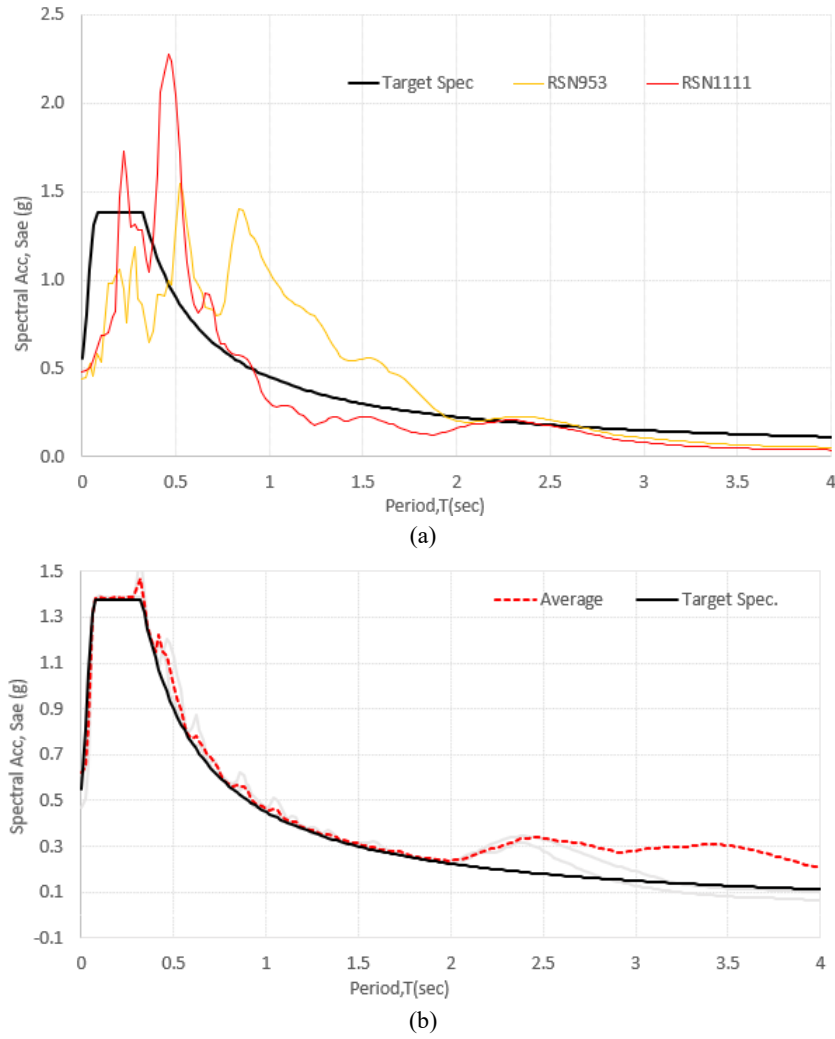


Fig. 24. Scaling of the ground-motion records: (a) Unscaled records; (b) Scaled records

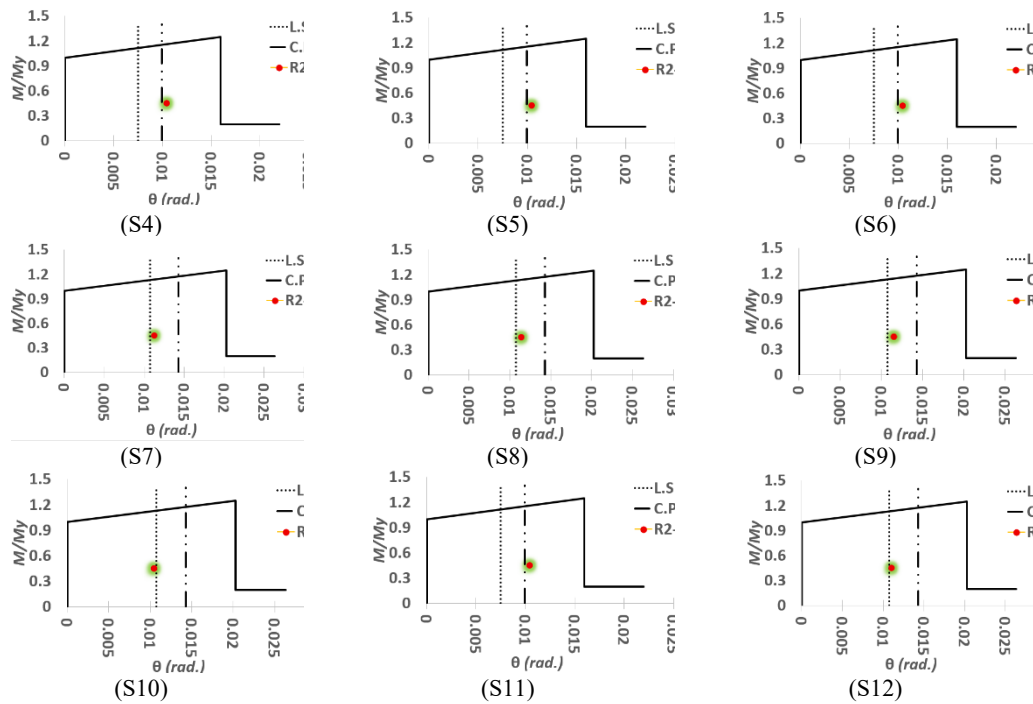


Fig. 25. Damage states of the ground-story columns of the reference structure based on the time-history analysis

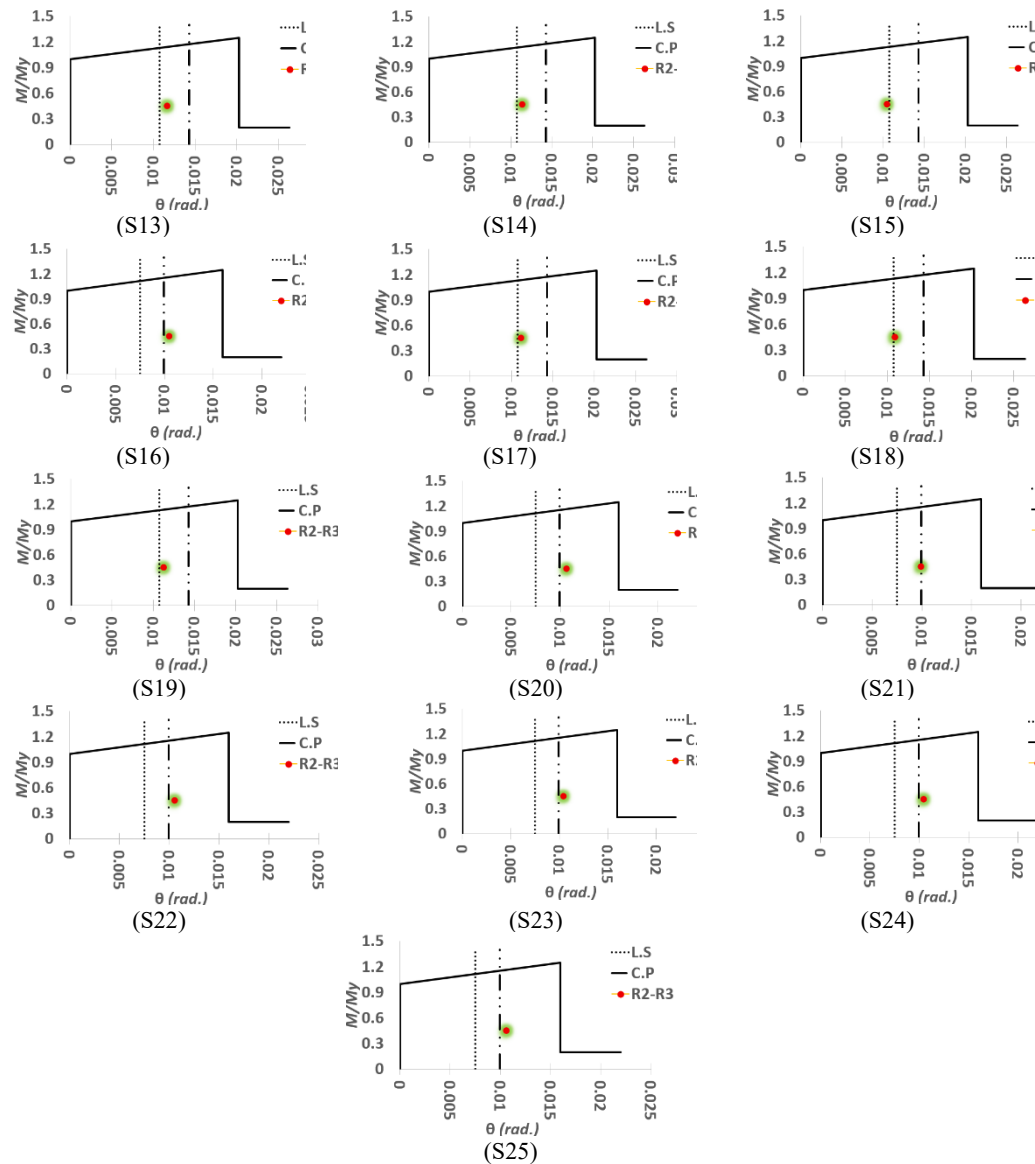


Fig. 25. Continued

Table 19. Damage states observed in the ground-floor columns according to the analysis method

Analysis Type	MD	AD	COLLAPSE
Pushover analysis	-	S7,S8,S9,S10, S12,S13,S14, S15,S17,S18, S19	S1,S2,S3,S4,S5,S6,S11,S16,S20,S21,S22,S23,S24,S25
Time history analysis	S10,S15	S7,S8,S9,S12,S13,S14,S17,S18,S19,S21	S1,S2,S3,S4,S5,S6,S11,S16,S20,S22,S23,S24,S25

To compare the analysis results obtained from the pushover and time-history analyses, the damage states of the ground-floor columns according to the analysis method are presented in Table 19.

According to Table 19, the S10 and S15 columns were classified as being at the (AD) level in the pushover analysis, whereas they were classified as being at the (MD) level in the time-history analysis. Furthermore, the S21 column, which was classified as being in the Collapse state in the pushover analysis, was classified as being at the (AD) level in the time-history analysis. As shown in Fig. 25, the S10 and S15 columns are located very close to the (LS) limit, while the S21

column is very close to the Collapse Prevention (C.P.) limit. Therefore, the results obtained from the two analysis methods can be considered to be in good agreement.

Although the time-history analysis method provides more realistic results, its application requires the selection of site-specific ground-motion records and analysis by qualified structural engineers. Moreover, this method provides substantially more extensive response data for both the dampers and the structural system. In the present study, time-history analysis was employed using a limited number of ground-motion records primarily to provide a rapid comparison with the pushover analysis results.

5.2. Time-history analysis of the friction damper retrofitted structure

The damage states of the ground floor columns of the damper-retrofitted structure were determined using time-history analysis, and all columns were found to be at the MD level. In the analyses, the total damper sliding force capacities were set not to exceed 25% of the average story base shear force in each direction [45]. The Plastic Wen method was used to define the dampers. The damper parameters given in Table 5 were used for the definition of the dampers.

Time-history analysis enables the hysteretic response of the dampers to be obtained. This hysteretic response also allows the damping ratio of the dampers to be calculated. The equivalent damping for the damper elements can be calculated using Eq. (13) [39].

$$\zeta_{eq} = \frac{1}{4\pi} \frac{E_{D0}}{E_{S0}} \quad (13)$$

where, E_{D0} represents the energy dissipated over the entire cycle, while E_{S0} represents the strain energy.

Fig. 26 shows the hysteretic response of a ground-floor damper, from which the equivalent damping ratio was calculated as 0.56 using Eq. (13). In accordance with TBEC 2018, this additional damping was not incorporated into the target or demand spectra.

The analyses conducted in this study provide detailed insights into the seismic performance of the structure; however, the analytical findings should be further validated against experimental evidence. Suk et al. [47] experimentally investigated the effects of friction dampers on RC frames under quasi-static loading using story drift ratios and TBEC 2018 damage criteria. At 1% drift, hairline cracks appeared in the reference frame, while the damper-retrofitted frame remained uncracked. At 2%, cracking widened in the reference frame and began to develop in the retrofitted frame. At 3%, the reference frame columns exhibited damage, whereas the retrofitted frame showed crack widening without significant load-carrying damage. Column rotations were also measured, and the corresponding damage states were evaluated according to TBEC 2018.

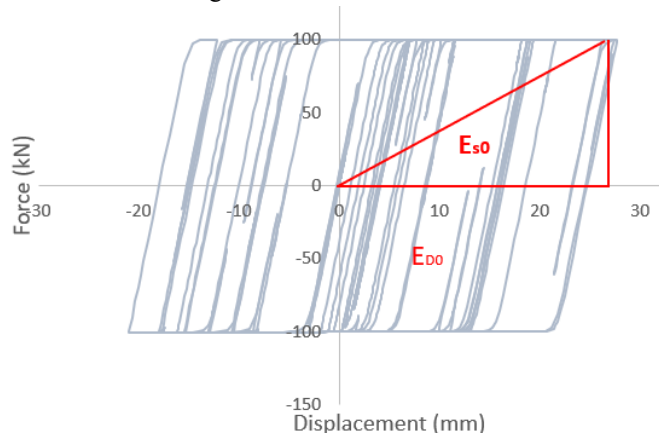


Fig. 26. Hysteretic curve of a damper with a 100 kN sliding capacity

6. Conclusions

For RC structures with inadequate seismic performance, friction dampers may be preferred when preserving architectural requirements, minimizing construction time, maintaining uninterrupted occupancy, and reducing overall retrofit costs are the primary considerations. In contrast, RC jacketing can be recommended as one of the most effective strengthening techniques for improving structural performance when structural intervention and temporary interruption of building use are acceptable. Nevertheless, practical retrofitting projects often involve both architectural constraints and limitations related to structural intervention, making it difficult for a single strengthening technique to satisfy all design requirements. Under such circumstances, hybrid retrofitting strategies become a more appropriate solution. In addition to all these considerations, the cost of retrofitting is one of the most important factors in the selection of a strengthening method.

In this study, RC jacketing and friction damper retrofitting techniques were employed both individually and in a hybrid configuration to enhance the seismic performance of reference reinforced concrete structures. The main findings of the study are summarized as follows:

- The reference structure results indicated that 50% of the ground- and second-floor beams were classified as (LD), while the remaining 50% were classified as (MD); all other beams were classified as LD. In addition, notably, 36% of the ground-floor columns were classified as (AD), while the remaining 64% were classified at the Collapse level. These findings indicate that the reference structure is in a collapse condition according to TBDY 2018.
- Regarding the reinforced-concrete jacketing strengthening method, all structural members were found to remain within the specified damage limits. In terms of the distribution of damage states across the stories, 40% of the columns on the ground and second floors were classified as (MD), while the remaining 60% were classified as (LD). All other columns remained at the LD level. For the beams, 70% of the ground floor beams were classified as LD and 30% as MD, while all other beams remained at the LD level. Accordingly, the structure was found to satisfy the (LS) performance requirement according to TBEC 2018.
- When the structure was retrofitted with dampers having shear capacities of 50 and 100 kN, 100% of the ground floor columns were classified as Moderate Damage (MD), while 100% of the second-floor and remaining columns were classified as Limited Damage (LD). Regarding the beams, 50% of those on the ground and upper floors were classified as Limited Damage (LD) and the remaining 50% as Moderate Damage (MD), whereas all beams on the

other floors remained at the LD level. Accordingly, the structure was found to satisfy the Life Safety (LS) performance level according to TBDY 2018.

- For the hybrid retrofitting approach, 40% of the ground floor columns were classified as (LD), while the remaining 60% were classified as (MD). At the second floor, 72% of the columns were classified as LD and 28% as MD, whereas all other columns remained at the LD level. In addition, 77.5% of the ground floor beams were classified as LD, while the remaining 22.5% were classified as MD. All beams on the second and remaining floors were classified as LD. Accordingly, the structure satisfied the (LS) performance requirement according to TBEC 2018.
- A comparison of the structural periods shows that the reference structure has a period of 2.244 s, while the structures retrofitted with friction dampers, reinforced-concrete (RC) jacketing, and the hybrid system have periods of 2.157 s, 1.752 s, and 1.740 s, respectively. Accordingly, the period of the friction damper-retrofitted structure is only 0.087 s lower than that of the reference structure, whereas the periods of the RC jacketed and hybrid structures are lower by 0.492 s and 0.504 s, respectively. These results indicate that RC jacketing provides a substantially greater increase in structural stiffness than friction damper retrofitting alone, as evidenced by the greater reduction in the fundamental period relative to the reference structure.
- Regarding the base shear forces, the reference structure exhibited a base shear force of 2811.76 kN, compared with 2892.79 kN for the structure retrofitted with friction dampers, 4269.95 kN for the RC jacketed structure, and 4212 kN for the hybrid-retrofitted structure. Taking the base shear force of the reference structure as unity, the corresponding values for the friction damper, RC jacketing, and hybrid retrofitting schemes were 1.029, 1.52, and 1.498 times that of the reference structure, respectively. These results particularly highlight the importance of evaluating the foundation system in RC jacketed structures, considering the increased base shear forces and the associated additional overturning moments induced by the strengthening intervention.
- The reference structure and the damper-retrofitted structure were also analyzed using the time-history analysis method with the Northridge-01 (1994) and Kobe (1995) earthquake records, and the results were found to be highly consistent with those obtained from the pushover analyses.
- At the foundation level, the moments obtained for the RC jacketing and hybrid retrofitting systems were 358% and 239% higher, respectively, than those of

the reference structure. In contrast, the corresponding increase in the structure retrofitted with friction dampers was only 6%. This finding indicates that structures strengthened using RC jacketing and hybrid retrofitting systems may require substantial foundation strengthening, consequently leading to a significant increase in the overall retrofitting cost.

The findings of this study demonstrate that, depending on architectural constraints, occupancy continuity requirements, and project-specific performance objectives, not only the individual application of friction dampers or RC jacketing but also their hybrid implementation constitutes an effective strategy for the seismic retrofitting of reinforced concrete structures. However, detailed analyses are required to determine the most economically advantageous option among these retrofit systems, as achieving both adequate seismic resistance and economic efficiency is a critical factor in the retrofitting of structures.

Declarations

Conflict of interests

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Author contributions

All authors contributed equally to this study. All authors have read and approved the final manuscript and accept equal responsibility for its content.

Data availability statement

The data presented in this study are available on request from the corresponding author.

Use of generative AI and AI-assisted technologies

The author(s) confirm the author(s) did not use any AI tools in the preparation of this work/research/study.

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