Performance Evaluation of Curved Damper Truss Moment Frames Designed Using Equivalent Energy Design Procedure

S. Dehghani¹, S. F. Fathizadeh¹, T.Y. Yang²✉, E. Noroozinejad Farsangi³, A. R. Vosoughi¹,
I. Hajirasouliha⁴, Ch. Málaga-Chuquitaype⁵, I. Takewaki⁶

¹ Department of Civil and Environmental Engineering, School of Engineering, Shiraz University, Shiraz, Iran

² Department of Civil Engineering, The University of British Columbia, Vancouver, Canada

³ Faculty of Civil and Surveying Engineering, Graduate University of Advanced Technology, Kerman, Iran

⁴ Department of Civil and Structural Engineering, The University of Sheffield, Sheffield, UK

⁵ Department of Civil and Environmental Engineering, Imperial College London, London, UK

⁶ Department of Architecture and Architectural Engineering, Kyoto University, Japan

Abstract

Curved damper truss moment frame (CDTMF) system is a novel seismic force-resisting structural system which utilizes the curved dampers integrated into a semi-rigid moment frame to dissipate the earthquake input energy. To ensure the CDTMF has high performance, the state-of-the-art equivalent energy design procedure (EEDP) is applied to its design. EEDP allows engineers to design CDTMF to achieve different performance objectives at different levels of earthquake shaking intensities. In this study, two different prototype buildings (three and nine-story) were designed using EEDP. The seismic performance of the buildings was assessed using nonlinear time history analysis (NTHA) and incremental dynamic analysis (IDA). The results of the nonlinear dynamic analyses demonstrate that the CDTMFs can achieve the predefined performance targets (selected by the designer) at different earthquake intensity levels. The results of the IDA also show that the EEDP designed CDTMF has a sufficient margin against collapse. The results of this study in general confirm that CDTMF can be used as an effective seismic force-resisting system.

KEYWORDS: Curved Damper; Energy Dissipation; EEDP; NTHA; IDA; Fused Structures; Passive Control.

1. Introduction

Current seismic design codes generally aim to design structures to achieve a collapse prevention performance objective through inelastic deformation of structural elements [1, 2]. These inelastic deformations may result in severe structural damage which could be difficult and costly to repair [3, 4]. In order to address these issues, in recent years, efficient design philosophies and innovative resilient seismic resisting systems have been developed [5-8]. The new seismic design approaches aim to limit the structural damage on specially designed structural components which can be easily repaired or replaced efficiently [9, 10]. These structural elements are sometimes called structural fuses. Well-designed structural fuses are designed to be decoupled from the gravity load-bearing system. Hence, they can be repaired or replaced without affecting the functionality of the whole structure [3, 5, 11, 12].

Leelataviwat et al. investigated the seismic design and performance of knee-braced frames (KBFs). The frames were designed so that the input seismic energy was dissipated through the yielding of knee braces and also the formation of plastic hinges at the end of beams [13]. Similarly, Junda studied the seismic

performance of the knee braced moment frames containing partially restrained connections [14]. Shin et al. also developed a new buckling-restrained knee bracing (BRKB) system using two steel channel sections restraining the core steel plate. The proposed BRKB system was tested in a one-story one-bay steel frame under cycling loading [15]. In order to overcome the limitations of moment steel frames in providing large spans, Yang et al. and Wongpakdee et al. proposed the buckling restraint knee braced truss moment frame (BRKBTMF). BRKBTMF utilizes buckling-restrained braces (BRBs) to dissipate the earthquake energy, while retaining the benefit of large interior span using steel trusses [11, 16]. The seismic behavior and design of knee-braced frames considering buckling-restrained braces (BRBs) and single plate shear connections (SPSCs) were investigated by Doung [17]. In another relevant study, Yang et al. developed a new metallic damper named Welded Wide Flange Fuse (WWFF), which dissipates the earthquake input energy through shear yielding of the web in welded wide flange sections [18]. Junda et al. evaluated the performance of buckling-restrained knee braces (BRKBs) through cycling tests. The results of their study showed that BRKBs can be used as a reliable alternative to conventional force-resisting systems [19]. In general, the main advantages of KBF systems include using simple hinge connections for construction purposes, their replaceability after severe earthquakes, and their ability to dissipate the major part of seismic energy in pre-determined elements. However, the plastic hinges are likely to be formed in beams and also, the BRBs are expensive devices.

In order to mitigate the abovementioned shortcomings, in this paper, a novel seismic force-resisting system named curved damper truss moment frame (CDTMF) is proposed. CDTMF combines the economic and structural benefits of steel trusses and curved dampers to enhance the seismic performance of steel structures. Curved damper (CD) is a curve-shaped laser-cut steel plate introduced by Hsu and Halim [20, 21] and its seismic performance in multi-story innovative toggle-brace-curved damper (TBCD) systems was investigated by Dehghani et al. [22]. The inclusion of the CD improves the performance of the semi-rigid steel frame by increasing the strength, stiffness, and energy dissipation capability of the system. The concept of the curved damper is shown in Fig. 1. Detailed information about the curved damper can be found in [20, 21].

Fig 1. The geometry of the curved damper [20].

In the CDTMF system, the input seismic energy is dissipated using CD as the primary structural fuses and semi-rigid connections as secondary structural fuses. As a result, the beams and columns remain mainly undamaged during an earthquake event. Furthermore, the CD is decoupled from gravity system, so it can be inspected quickly and replaced easily after severe earthquakes. In order to demonstrate the efficiency and feasibility of the proposed system, two CDTMF structures (three and nine-story) are designed based on the state-of-the-art equivalent energy design procedure (EEDP) [5]. EEDP allows engineers to design structural fuse systems to achieve different performance objectives at different hazard levels of earthquake shaking intensities. Detailed finite element models of the CDTMF were developed using OpenSees software [23]. The seismic performance of CDTMF was investigated using both nonlinear time history analysis (NTHA) and incremental dynamic analysis (IDA) [24] according to the requirements of FEMA

P695 [25]. The results show that the proposed CDTMF can be an efficient seismic force-resisting system with excellent performance under different earthquake intensity levels.

2. Curved damper truss moment frame (CDTMF) system and the energy dissipation mechanism

In this study, an innovative, resilient, and cost-effective curved damper truss moment frame (CDTMF) system, shown in Figures 2 and 3, is introduced as an alternative seismic force-resisting system. As shown in Fig. 2, in the proposed system the curved dampers are connected to the truss structure via pinned connections. The connection between the truss members and the top chord beam is the same as truss structures. The CDTMF benefits from two designated yielding fuses. The first fuse is the CD and the second fuse is the semi-rigid (SR) connections. These structural fuses are designed to dissipate the earthquake energy, while the beams and columns are capacity designed to remain elastic. To be more precise, when low-intensity earthquakes happen, the curved dampers are designed to yield first, whereas the columns, beams, trusses and semi-rigid connections are set to remain elastic. At stronger hazard levels, the semi-rigid connections play the role of secondary fuses and given their relative stiffness and strength, they start yielding and dissipating the input seismic energy through their nonlinear rotational capacity. After an earthquake event, these structural fuses can be repaired or easily replaced without affecting the functionality of the building. Hence, the CDTMF can remain functional shortly even after strong earthquakes. In the proposed system, the truss-beam assembly plays some important roles; 1) It provides enough rigidity to transfer the forces and deformations to the primary structural fuses, while it prevents the beam from yielding; 2) This combination makes it possible for the CDTMF to span large openings; 3) In the proposed CDTMF system, the curved dampers are connected to truss members at each end. Therefore, the high concentrated shear forces transferred to the top chord beam are distributed and hence reduced.

Fig 2. The geometry of the proposed CDTMF system

Fig 3. The scenario for implementing CDTMF in the structure.

Fig. 4 illustrates the idealized trilinear lateral force-deformation response of fused structures introduced by Wongpakdee et al. [5, 16] and Yang et al. [5, 16]. At the service level earthquake (SLE), the structure is expected to remain elastic to achieve the Immediate Occupancy (IO) performance objective. At the design-based earthquakes (DBE), the primary fuses (CD) are designed to yield and dissipate energy, while the beams, trusses, columns and secondary fuses (SR) are designed to remain elastic. Since the CDs are designed not to carry gravity load, they can be efficiently inspected and replaced if needed. This makes the CDTMF capable of being recovered rapidly after strong earthquake events. Therefore, CDTMF can achieve the Rapid Return (RR) performance objective at the DBE shaking intensity. At the maximum credible earthquakes (MCE), primary (CD) and secondary fuses (SR) contribute to the energy dissipation to prevent the structure from collapse, so the structure can achieve Collapse Prevention (CP) performance objective.

Fig 4. The schematic trilinear lateral response of the CDTMF system.

To achieve the designed performance objectives, Yang et al. (2018) [5] developed the equivalent energy design procedure (EEDP). Fig. 5 depicts the concept of EEDP. The vertical axis represents the base shear, F , which can be calculated using the spectra acceleration, S_a , multiplied by the structural mass, m . The horizontal axis represents the roof drift ratio, Δ , which can be calculated using the spectra displacement, S_d , multiplied by the C_0 coefficient. Details of the C_0 can be identified in ASCE-7 [26].

EEDP allows designing the structure to achieve different performance objectives at different hazard levels. EEDP assumes that the energy dissipated by an equivalent nonlinear single-degree-of-freedom (ENLSDOF) system (elastic strain energy, E_s , and hysteretic energy, E_h) equals to the equivalent linear single-degree-of-freedom (ELSDOF) system (elastic energy, E_a). The procedure to design CDTMF using EEDP has been summarized in the following steps. Detailed information about the EEDP design method can be found in [5].

Fig 5. Concept of the EEDP design approach

- **Step 1: Select seismic hazard levels**

Designers shall consult the owner and/or available design codes to identify the SLE, DBE and MCE shaking intensities. At the SLE shaking intensity, the structure shall be design to be elastic. At the DBE shaking intensity, the primary fuses (CDs) are designed to yield, while the rest of the structure is designed to remain elastic. At the MCE shaking intensity, both the primary fuses (CDs) and secondary fuses (SRs) are designed to yield to ensure the CDTMF can achieve the Collapse Prevention (CP) performance objective.

- **Step 2: Calculate F_y and T**

Up to the SLE hazard level, the CDTMF is designed to remain elastic. The designer shall select the maximum yield displacement ratio, Δ_y , up to which the structure will remain elastic. Once the Δ_y is selected, the F_y can be obtained from the intersection of the SLE hazard curve and Δ_y . In addition, the natural period of the structure, T can be obtained according to Eq. (1):

$$T = 2\pi \sqrt{\frac{\Delta_y H / C_0}{F_y / m}} \quad (1)$$

where H is structure's height, and m is the [effective seismic](#) mass.

- **Step 3: Calculate F_p**

At the DBE hazard level, designer shall select, Δ_p , which represents the plastic roof drift ratio of the system, where the secondary fuses are designed to yield. Once Δ_p has been selected, the plastic base shear, F_p , can be calculated according to Eq. (2):

$$F_p = \frac{2\Delta E_{E1}}{\gamma_a H (\Delta_p - \Delta_y)} - F_y \quad (2)$$

$$\gamma_a = \frac{2\mu_s - 1}{R_\mu^2}$$

where γ_a is the energy modification factor, μ_s is the ductility factor, and R_μ is the ductility reduction factor which are all presented by Yang et al (2018) [5]. ΔE_{E1} is the incremental energy of the ENLSDOF between the SLE and DBE hazard levels as shown in Eq. (3):

$$\Delta E_{E1} = \frac{mC_0}{2} [(S_a)_{DBE} - (S_a)_{SLE}] [(S_d)_{DBE} - (S_d)_{SLE}] \quad (3)$$

where $(S_a)_{DBE}$ and $(S_a)_{SLE}$ are the spectral acceleration and displacement at DBE and SLE levels, respectively. $(S_d)_{DBE}$ and $(S_d)_{SLE}$ are the spectral displacement at DBE and SLE levels, respectively.

- **Step 4: Calculate Δ_u**

In order to prevent the CDTMF structure from collapse at the MCE hazard level, the primary and secondary fuses should be designed to maintain their yielding strength until the structure meets the ultimate roof drift ratio, Δ_u , which is calculated using Eq. (4):

$$\Delta_u = \frac{\Delta E_{E2}}{\gamma_b H F_p} - \Delta_p \quad (4)$$

where γ_b is the energy modification factor and ΔE_{E2} is the incremental energy of the ENLSDOF between the DBE and MCE hazard levels which can be calculated using Eq. (5):

$$\Delta E_{E2} = \frac{mC_0}{2} [(S_a)_{MCE} - (S_a)_{DBE}] [(S_d)_{MCE} - (S_d)_{DBE}] \quad (5)$$

where $(S_a)_{MCE}$ and $(S_d)_{MCE}$ are the spectral acceleration and displacement at MCE level, respectively.

- **Step 5: Distribute base shear between primary and secondary seismic resisting force systems**

Once the trilinear force-deformation relationship of the system has been defined in step 1 to 4, the yielding and plastic base shears should be distributed between the primary and secondary fuses based on the relationships presented in Eq. (6) [5]:

$$F_{PR} = F_y \frac{\mu - \lambda}{\mu - 1} \quad , \quad F_{SE} = F_y \mu \frac{\lambda - 1}{\mu - 1} \quad (6)$$

where F_{PR} and F_{SE} are the yielding strengths of primary and secondary fuses, respectively; $\mu = \frac{\Delta_p}{\Delta_y}$ and $\lambda = \frac{F_p}{F_y}$.

Finally, the abovementioned forces should be distributed over the building height according to following relationships (Eq. (7-8)):

$$F_{PR,i} = C'_{vi} F_{PR} \quad , \quad F_{SE,i} = C'_{vi} F_{SE} \quad (7)$$

where $F_{PR,i}$ and $F_{SE,i}$ are the story shear distributed between the primary and secondary seismic resisting force systems, respectively.

$$C'_{vi} = (\beta_i - \beta_{i+1}) \left(\frac{w_n h_n}{\sum_{j=1}^n w_j h_j} \right)^{0.75T-0.2} \quad , \quad \beta_i = \left(\frac{\sum_{j=1}^n w_j h_j}{w_n h_n} \right)^{0.75T-0.2} \quad (8)$$

where w_n and w_j are the seismic weights of floor and level j ; h_n and h_j are the height of floor and level j .

- **Step 6: Distribute base shear between primary and secondary seismic resisting force systems**

Once the fuses have been designed, the beam, column and foundation are capacity designed to remain elastic.

3. Prototypes design and numerical modelling approach

3.1. Prototypes design

In this study, two prototype buildings (three and nine-story) located in downtown LA are adopted from [27]. Figures 6 and 7 show dimensions of the models. The bays width of three-story prototype is 9.14 m and the story height is 3.96 m. The height and width of floors and bays in nine-story prototype are 4.13 m and 9.14 m, respectively. The soil condition is assumed to be type C. The SLE, DBE, and MCE seismic hazards are selected as 87%, 10% and 2% probability of exceedance in 50 years for the prototype site. Fig. 8 shows the design spectrum for the prototype building. Table 1 shows the design value for the loads adopted from [27].

Table 2 shows EEDP design parameters for the prototype building. In order to concentrate the nonlinearity in the primary and secondary fuses and achieve the two-phased energy dissipation mechanism according to the concept of EEDP, the columns, beams and trusses were capacity-designed to remain elastic. This

procedure was thoroughly explained in [5, 12]. The dimensions of the curved dampers and characteristics of semi-rigid connections are obtained based on EEDP to meet various performance objectives at different hazard levels. Tables 3-5 show the properties of the structural members for the prototype buildings. All the materials are assumed to have a modulus of elasticity (E) of 210000 MPa, while the yield stress of columns, beams and dampers are selected as 345, 248 and 293 MPa, respectively. The length and angle (related to the column, α) of the curved dampers are set to be 1.0 m and 27° , respectively. Also, the curved damper angle (θ) is set to be 60° (Fig. 9).

Fig 6. Geometry of three-story CDTMF structure

Fig 7. Geometry of nine-story CDTMF structure

Fig 8. The selected hazard levels [28].

Table 1. Loading and seismic weight properties of the structures

Story	Three-story	
	Gravity load (KN/m)	Seismic Weight (KN)
1-2	32.0	4688
Roof	28.7	5071
	Nine-story	
	Gravity load (KN/m)	Seismic Weight (KN)
1-8	32.0	4857
Roof	28.7	5231

Table 2. EEDP design parameters for structures

	Δ_y	Δ_p	Δ_u	C_0	γ_a	γ_b
3 story	0.38 %	1.30%	2.40%	1.3	1.85	2.3
9 story	0.36%	1.10%	2.00%	1.4	2.00	2.05

Table 3. Cross-sectional properties of the structures

Section	Three-story	Nine-story
C1	W14X257	W14X370
C2	W14X311	W14X283
C3	-	W14X257
C4	-	W14X233
C5	-	W14X500
C6	-	W14X455
C7	-	W14X370
C8	-	W14X283
C9	-	W14X257
B1	W33X118	W36X160
B2	W30X116	W36X135
B3	W24X68	W30X99
B4	-	W27X87
B5	-	W24X68

Table 4. Cross-sectional properties of truss elements

Story	Three-story			Nine-story		
	Horizontal	Vertical	Diagonal	Horizontal	Vertical	Diagonal
1	2L6X6X7/8	2L4X4X3/4	2L6X6X5/8	2L6X6X1	2L5X5X5/8	2L6X6X3/4
2	2L6X6X7/8	2L4X4X3/4	2L6X6X5/8	2L6X6X1	2L5X5X5/8	2L6X6X3/4
3	2L6X6X7/8	2L4X4X3/4	2L6X6X5/8	2L6X6X1	2L5X5X5/8	2L6X6X3/4
4	-	-	-	2L6X6X7/8	2L4X4X5/8	2L6X6X9/16
5	-	-	-	2L6X6X7/8	2L4X4X5/8	2L6X6X9/16
6	-	-	-	2L6X6X7/8	2L4X4X5/8	2L6X6X9/16
7	-	-	-	2L6X6X3/4	2L3X3X1/2	2L5X5X3/8
8	-	-	-	2L6X6X3/4	2L3X3X1/2	2L5X5X3/8
9	-	-	-	2L6X6X3/4	2L3X3X1/2	2L5X5X3/8

Table 5. The obtained sizes of the curved dampers and the characteristics of the semi-rigid connections.

Story	Three-story				Nine-story			
	Curved damper		Connection		Curved damper		Connection	
	d (cm)	t (cm)	α^*	β^{**}	d (cm)	t (cm)	α^*	β^{**}
1	20.0	5.0	3.0	0.60	20.0	5.0	2.0	0.40
2	20.0	5.0	2.5	0.50	20.0	5.0	2.8	0.40
3	16.0	5.0	2.0	0.35	24.0	6.0	2.8	0.35
4	-	-	-	-	24.0	6.0	2.8	0.35
5	-	-	-	-	20.0	5.0	2.6	0.40
6	-	-	-	-	20.0	5.0	2.6	0.40
7	-	-	-	-	19.0	5.0	2.5	0.40
8	-	-	-	-	19.0	5.0	2.5	0.40
9	-	-	-	-	14.0	4.0	2.0	0.30

* The ratio of initial stiffness of connection to elastic bending stiffness of the connected beam.

** The ratio of plastic-moment capacity of connection to plastic-moment capacity of the connected beam.

Fig 9. The geometric characteristics of the CDTMF system.

3.2. Numerical modelling approach

The selected prototype structures are modelled in OpenSees [23]. Beams and columns are modelled using nonlinear beam-column elements with fiber sections. The truss members are assumed to remain elastic. P- Δ effects are taken into account using the P-Delta geometric transformation. The steel's stress-strain relationship is defined using the uniaxial Steel02 material with a strain hardening ratio of 2%. Zero-length spring element with Steel02 material in OpenSees is used to model the semi-rigid connections. The curved dampers are modeled using nonlinear beam-column elements with Steel02 material and corotational geometric transformation. The Minmax material in OpenSees is used to constrain the stress-strain response

of the CDs and SRs. More information about the details of parameters in the adopted behavioral models can be found in Appendix 1.

To demonstrate the accuracy of the utilized modeling approach, a one-story one-bay semi-rigid frame and also a curved damper were modeled using the Steel02 material and fiber sections. The height and bay of the test frame were 2.52m and 4.744m, respectively. The columns and beams' profiles in the semi-rigid frame were $H250 \times 250 \times 9 \times 14$ and $H200 \times 200 \times 8 \times 12$, while the height and width of the curved damper section were 9cm and 3cm, respectively. More information about the test specimens can be found in [20, 21]. Fig. 10 shows the detailed of the specimens modeled numerically and the comparison of the force deformation response of the CD and SR against available experimental results. The results clearly demonstrate that the adopted modeling approach can efficiently simulate the nonlinear response of the semi-rigid frame as well as the CD and SR [20, 21].

Fig 10. The validation of numerically modeled curved dampers against the reference experimental tests [20, 21].

The mass and gravity loads were assigned to the nodes using the tributary area. 2.5% Rayleigh mass and stiffness proportional damping was assigned to the first and third vibration mode. Table 6 shows the periods associated with the first 3 vibration modes of the prototype buildings.

Table 6. The results of modal analysis for three- and nine-story prototypes

Modes	Three-story (sec)	Nine-story (sec)
1 st	0.67	1.65
2 nd	0.23	0.61
3 rd	0.13	0.36

The obtained pushover curves and their equivalent trilinear curves are illustrated in Fig. 11. As can be seen, the obtained trilinear capacity curves meet well the EEDP's design assumptions listed in Table 2. The curved dampers (primary fuses) and semi-rigid connections (secondary fuses) start to yield at the designated Δ_y and Δ_p , respectively. Furthermore, the obtained Δ_u in both CDTMF structures indicate that the structural fuses satisfy the calculated values by EEDP. This demonstrates the robustness of the EEDP design.

Fig 11. The obtained capacity and their equivalent trilinear curves.

3.3. Nonlinear time-history analyses

In order to perform the nonlinear dynamic analyses of the CDTMF prototypes, a set of twenty-two far-field ground motions with distances greater than 10 km from fault rupture is selected from FEMA P695 [25]. All earthquake records have a magnitude of more than 6.5 Mw and are recorded on soil types C or D. The ground motions are then scaled so that their median spectrum matches the response spectra of SLE, DBE and MCE hazard levels between the period range of $0.2T$ and $1.5T$, where T is the fundamental period of the structure [26]. Figures 12 and 13 show the scaled ground motions spectra for the three-story and nine-story CDTMF structures, respectively.

Fig 12. Response spectra for three-story structure.

Fig 13. Response spectra for nine-story structure.

Fig. 14 shows the distribution of maximum roof drift ratio for the three- and nine-story CDTMF structures. Fig. 14.a shows that the median of roof drift ratios (RDRs) for three-story CDTMF structure are 0.34%, 1.23% and 2.34% at SLE, DBE and MCE hazard levels, respectively. The corresponding values for nine-story structure (Fig. 14.b) are 0.29%, 1.05% and 1.92% at SLE, DBE and MCE hazard levels, respectively. These values matched with the target RDRs selected by EEDP.

Fig 14. Roof drift ratio probability of exceedance.

The forces and deformations in trusses, beams and columns were investigated by post-processing the results. The maximum forces in truss elements indicate that they remained elastic. Figures 15 and 16 show the demand capacity ratio (DCR) of the CDs and SRs recorded during the selected earthquakes at different hazard levels for the three- and nine-story CDTMF structures, respectively. The yield and ultimate strain of the curved dampers are set as 0.004 and 0.065 according to Hsu and Halim [21]. The results show that at the SLE hazard level, both primary (CD) and secondary (SR) fuses remain elastic. This indicates that the CDTMF has achieved the IO performance level. At the DBE hazard level, the primary fuses (CD)

exhibited yielding, while secondary fuses (SR) remained elastic. Since the primary fuses are designed to be easily replaced, the CDTMF is expected to achieve the RR performance level in this case. At the MCE shaking intensity, both the primary fuses (CD) and secondary fuses (SR) yielded to protect the CDTMF from collapse, while the global deformation was controlled. Therefore, the CDTMF is deemed reaching the CP performance objective under this earthquake intensity level.

Fig 15. Performance of primary (a-c) and secondary (d-f) fuses for three-story structure.

Fig 16. Performance of primary (a-c) and secondary (d-f) fuses for nine-story structure.

3.4. Collapse assessment

In order to ensure the CDTMF has sufficient capacity against collapse, the collapse margin ratio (CMR) for the CDTMF was assessed using the incremental dynamic analysis (IDA) procedure as outlined in FEMA P695 [25]. The maximum inter-story drift ratio (ISDR) of 10% is defined as the collapse criterion. Fig. 17 shows the results of the IDA analysis. A log-normal cumulative probability distribution function is considered to obtain the fragility curves (Eq. 9). The results show that the CMR for three- and nine-story CDTMF structures are 2.72 and 2.39, respectively. According to FEMA P695, adjusted CMR (ACMR) is calculated by accounting for the spectral shape factor (SSF). The ACMRs are 3.2 for three-story and 3.23 for nine-story prototypes, respectively. To verify acceptable structural safety against collapse, the calculated ACMR should be greater than the allowable ACMR10% provided by FEMA P695 for different exceedance levels and various levels of uncertainty. The value of total uncertainty (β_{TOT}) for both structures are assumed as 0.726. The obtained ACMR10% for three- and nine-story structures is 2.53. The results show that CDTMFs designed using EEDP have sufficient capacity against collapse.

$$P(D \geq C|IM) = \phi\left(\frac{\ln S_D/S_C}{\beta_{tot}}\right) \quad (9)$$

where C is the drift capacity, D is the drift demand, S_D is the median of demand and S_C is the median of the selected limit state.

Fig. 17. The obtained IDA (a-b) and fragility curves (c-d).

4. Summary and conclusion

In this study, a novel and resilient seismic force-resisting system named curved damper truss moment frame (CDTMF) system was proposed. CDTMF utilizes curved dampers and semi-rigid connections as primary and secondary structural fuses to dissipate earthquake input energy, while the beams, trusses, columns and foundation are capacity designed to remain elastic. To ensure CDTMF has a high performance, the state-of-the-art equivalent energy design procedure (EEDP) was used to design CDTMF. A detailed design procedure for the CDTMF using EEDP is presented. To demonstrate the efficiency of the proposed system, two prototype buildings (three- and nine-story) were designed. Detailed finite element models were developed using OpenSees. The performance of the prototype buildings was assessed using nonlinear time history analysis (NTHA) and incremental dynamic analysis (IDA). It was shown that at the service level earthquake (SLE) the entire CDTMF remained elastic, which indicated the Immediate Occupancy (IO) performance objective was achieved. At the DBE shaking intensity, the curved dampers (CD) yielded to dissipate the earthquake energy, while the other structural elements remained elastic. Since the CDs were designed to be decoupled from the gravity load-bearing system, they can be easily inspected and replaced. This makes the CDTMF easy to be repaired, which means the CDTMF has reached the RR performance objective under DBE intensity level. At the maximum credible earthquake

(MCE), both the semi-rigid connections and the curve dampers dissipated sufficient energy to prevent the CDTMF from collapsing, hence allowing the CDTMF to reach the collapse prevention (CC) performance objective. Detailed incremental dynamic analyses were also conducted to assess the safety of CDTMF. The results of the analyses showed that CFTMF has a sufficient margin against collapse, which indicated CDTMF can be used as an efficient structural system for seismic applications.

Conflict of interest

The authors declare that they have no conflict of interest.

Data Availability Statement

Some or all data, models, or codes that support the findings of this study are available from the corresponding author upon reasonable request.

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Appendix:

The detailed parameters of the behavioral models are presented as follows:

Beam and Columns:

```
uniaxialMaterial Steel02 $matTag $Fy $E $b $R0 $cR1 $cR2 <$a1 $a2 $a3 $a4 $sigInit>
uniaxialMaterial Steel02 $matID $Fy 2.e11 0.02 20 0.925 0.15 0.0005 0.01 0.0005 0.01
```

Trusses:

```
uniaxialMaterial Elastic $matTag $E
uniaxialMaterial Elastic $matID_Truss 2.e11
```

Semi-rigid Connections:

```
uniaxialMaterial Steel02 $matTag $Fy $E $b $R0 $cR1 $cR2 <$a1 $a2 $a3 $a4 $sigInit>
uniaxialMaterial Steel02 $matID $β $α 0.02 10 0.925 0.50 0.004 0.01 0.004 0.01

uniaxialMaterial MinMax $matTag $otherTag <-min $minStrain> <-max $maxStrain>
uniaxialMaterial MinMax $matID $matID_Spr -min -0.06 -max 0.06
```

Curved dampers:

```
uniaxialMaterial Steel02 $matTag $Fy $E $b $R0 $cR1 $cR2 <$a1 $a2 $a3 $a4 $sigInit>
uniaxialMaterial Steel02 $matTag $Fy_D 2.e11 0.02 20 0.905 0.1 0.05 0.95 0.05 0.95

uniaxialMaterial MinMax $matTag $otherTag <-min $minStrain> <-max $maxStrain>
uniaxialMaterial MinMax $matID_D $matTag -min -0.065 -max 0.065
```