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Seismic Behavior of Steel Frames Equipped with Comb-Teeth Metallic Yielding Dampers

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Abstract

Comb-teeth damper (CTD), is a new type of metallic yielding damper, which is made of steel plates and includes a number of teeth that dissipate energy through in-plane flexural yielding. The behavior of individual samples of CTD have been previously studied numerically and experimentally and it has been shown that this damper has excellent energy dissipating capacity and large ductility ratio. In this paper, application of this type of damper to steel frames is studied. Sample steel frames are constructed and equipped with CTDs and tested under cyclic loading. The results show that these dampers can serve their intended duties and dissipate considerable amount of energy. Numerical modelling of the frames confirms the experimental results and shows that by correct proportioning of the members, frame members i.e. beams, columns and braces remain elastic during lateral loading. This allows using the CTDs as a replaceable energy dissipating device. Finally CTDs are included in a reference frame and their effects on reducing seismic demand are studied using non-linear time history analysis. The results show that by using a smaller volume of steel in CTD dampers compared to traditional TADAS, the same level of response reduction may be achieved, while utilizing economic advantage of this type of damper.

Keywords Passive seismic control · Metallic yielding dampers · Energy dissipation · TADAS dampers · Slit dampers

1 Introduction

The traditional approach to seismic design of buildings has been based on providing a combination of strength and ductility in main members of the structure to resist the imposed loads. However, it is well known that while this approach helps to prevent collapse of buildings and saves inhabitants lives, some levels of structural and non-structural damage are inevitable during severe earthquakes. Hence after such events, the building may no longer be used and they have to be demolished and rebuilt, which involves considerable costs. Therefore, nowadays the attention has been shifted to design structures in which, damages during intense earthquakes are minimized and concentrated in specific parts, so that the buildings remain operational after the earthquake and can be repaired by replacing affected elements without the need to demolish and reconstruct the whole building. In order to achieve the above goal, structural control techniques are being increasingly used in recent years. In this approach, the main objective of structural engineers is to reduce the structural response through restricting the input energy or increasing the energy dissipation capacity of structure. In a broad classification, these types of seismic mitigation methods can be divided into three general groups: base isolation, active or semi-active control and passive control of systems (Soong and Spencer 2002).

Among the above mentioned methods, passive energy dissipation devices have been widely used in structures as effective and relatively low-cost systems to reduce the earthquake damage. Energy dissipation can be achieved using various types of devices. Yielding of material in metallic dampers (Kelly et al. 1972), friction between surfaces in friction dampers (Pall and Marsh 1982; Filiatrault et al. 2000; Xu et al. 2001; Grigorian et al. 1993), flow of viscous fluids through narrow orifices in viscous dampers (Constantinou

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and Symans 1993; Vargas and Bruneau 2007; Palermo et al. 2013; Gidaris and Taflanidis 2014) and deformation of viscoelastic materials in viscoelastic dampers (Chang et al. 1994; Shen et al. 1995; Lai et al. 1995; Miranda et al. 1998), are some alternative mechanisms, which may be used to dissipate seismic energy. However, it may be said that manufacturing and installing metallic yielding dampers is normally simpler compared to other types of passive systems and so, they have more potential for widespread application in typical building constructions.

The initial researches on metallic yielding dampers is the studies of Kelly et al. (1972) and Skinner et al. (1975) and following ones which was done by Skinner et al. (1980) and Aiken and Kelly (1992). Well-designed metallic yielding dampers can be used for dissipating the seismic input energy through inelastic deformation of ductile metals in flexural, axial, shear or torsional modes. Added Damping and Stiffness, ADAS, and Triangular-ADAS, TADAS, systems (Bergman and Goel 1987; Xia and Hanson 1992; Tsai et al. 1993; Perry et al. 1993; Aiken et al. 1993) and also buckling restrained braces, BRBs, (Clark et al. 1999; Wada and Nakashima 2004; Tremblay et al. 2006; Takeuchi et al. 2008) are the most common types of yielding dampers. ADAS and TADAS dampers are flexural yielding dampers, which consist of a number of parallel X or triangular-shaped steel plates, respectively. In the BRBs, in order to prevent axial buckling of brace, the steel core is enclosed by a concrete casing and so both in compression and tension, the steel core can yield under axial loads and dissipate input energy effectively. In recent years, researches on metallic yielding dampers was followed and some other concepts for yielding metallic dampers have also been introduced. Among these researches the works of Williams and Albermani (2003), Hitaka and Matsui (2003), McCloskey (2006), Oh et al. (2009), Franco et al. (2010), Maleki and Bagheri (2010), Chana et al. (2013), Deng et al. (2014), Gray et al. (2014), Benavent-Climent (2010), Benavent-Climent et al. (2015) and Deng et al. (2015) can be mentioned. Some of the more developed types of yielding dampers are slit dampers that consist of parallel plates with a number of slits/openings (Li and Li 2007; Chana and Albermani 2008; Ma et al. 2010; Ghabraie et al. 2010; Lee et al. 2015). The slits/openings divide the steel plate into a series of links acting in flexure under the global in-plane shear deformation of damper.

Considering the results and observations of previous studies on slit dampers, Garivani et al. (2016) have recently presented a new metallic yielding damper named comb-teeth damper, CTD, which consists of a series of steel teeth acting in parallel and dissipating energy through in-plane flexural yielding deformation. The behavior of this damper has been verified through nonlinear finite element analyses and experimental studies of physical specimens tested under cyclic loads. Since the CTD is a new damper, it is necessary to evaluate the challenges of the performance and installation of this type of damper in a real frame. Therefore, in this paper, the behavior of steel frames equipped with CTDs is studied experimentally and numerically under cyclic loading. The results can confirm that CTD can be used in real braced frames as an effective energy dissipation device. Also elastic behavior of other structural frame members (beams, columns and braces) may confirm that the dampers may act as a good fuse and the frame can be reusable if the yielded dampers are replaced. Finally, in order to assess feasibility and economy of utilizing these dampers in structures, a frame equipped with TADAS, which has been reported in literature, is considered and TADAS dampers are replaced by CTDs. The seismic behavior of this frame in these two cases are studied through non-linear dynamic analysis and compared. The results can show whether or not this new damper has economic advantages.

Based on the above in Sect. 2 of this paper, the CTD is briefly described with reference to the previous work of Garivani et al. (2016). Experimental studies on steel frames equipped with CTD are explained in Sect. 3 and the numerical studies are presented in Sect. 4. Section 5 investigates the seismic response of framed structures equipped with CTD. Finally the conclusions of this research are provided in Sect. 6.

2 Description of Comb-Teeth Damper

Comb-teeth damper (CTD) is a new type of metallic yielding damper developed by Garivani et al. (2016) which is installed between floor beam and Chevron bracing in a building frames as shown in Fig. 1a. It is geometrically similar to half of a slit damper and can dissipate seismic energy through in-plane flexural yielding of its teeth (Fig. 1b). However, this design breaks up the strict connectivity of the teeth to each other at one end and allows controlling their axial and lateral deformations separately. From energy dissipation point of view, the optimum shape of teeth is the one that allows distributing induced inelastic deformations within the volume of material as evenly as possible. Based on this principle, Garivani et al. (2016) designed the geometry of the damper teeth in such a way that the magnitude of normal stresses caused by bending of plate at any section is independent of the distance from the end of teeth. The designed shape of CTDs and its geometric characteristics are shown in Fig. 1c. This shape follows a parabolic function as below

$$b(x) = 2\lambda\sqrt{x} \tag{1}$$

where the shape coefficient, λ, is a constant. Numerical and experimental study of CTD specimens have been conducted by Garivani et al. (2016) and the results have shown simultaneous yielding of outer fiber of the teeth along their length and a uniform spreading of yielding towards inner fibers without any strain concentration.

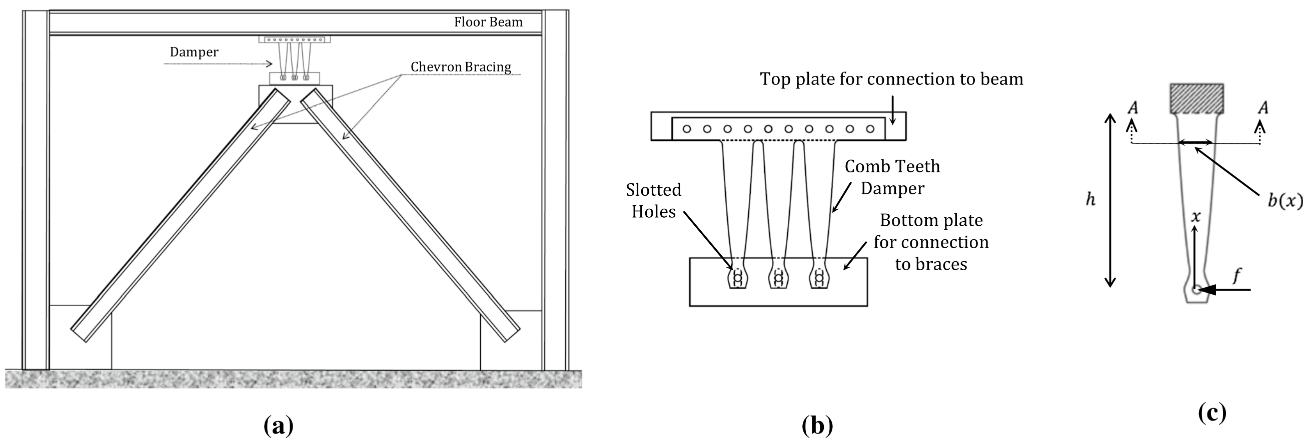


Fig. 1 A typical comb teeth damper in a frame. **a** Frame configuration, **b** components of a CTD, **c** individual links of the damper

3 Experimental Study of Steel Frames Equipped with Comb-Teeth Damper

In this research two simple steel frames equipped with comb-teeth damper were constructed and tested to study the performance of CTDs when they are implemented in frames structures. Figure 2 shows the experimental setup used in these tests. The tested frames, had dimensions, which are shown in Fig. 4. The bottom end of frame columns were connected to the strong floor using pin connections. Due to dimension limitations caused by the size of strong floor and reaction frames, tests were carried out on a half-scale steel frame. Since it was expected that the stiffness of simple steel frame would be negligible compared to the stiffness of combined braces and dampers, dampers were made full scale and installed in the frame. So basically the test frame was only geometrically half scale in length and the members section were full scale.

Cyclic horizontal loads were applied to the frame at the level of top floor beam using two hydraulic jacks. In order to restrain the out-of-plane deformations of the frame, constraining elements were fixed to the reactions frames on both sides of the frame. To measure the frame and damper deformations, five LVDTs were used, locations of which are shown in Fig. 2.

Based on the design philosophy of this type of structures, beams, columns and braces were designed so that under application of lateral loads only the dampers would yield and other members remain elastic. Accordingly, the cross sections of beams, columns and braces were IPE270, IPB120 and 2UNP80, respectively. The gusset plates were designed based on the tension capacity of braces.

A view of a tested frame is shown in the Fig. 3. In order to prevent out-of-plane displacements of damper and braces,

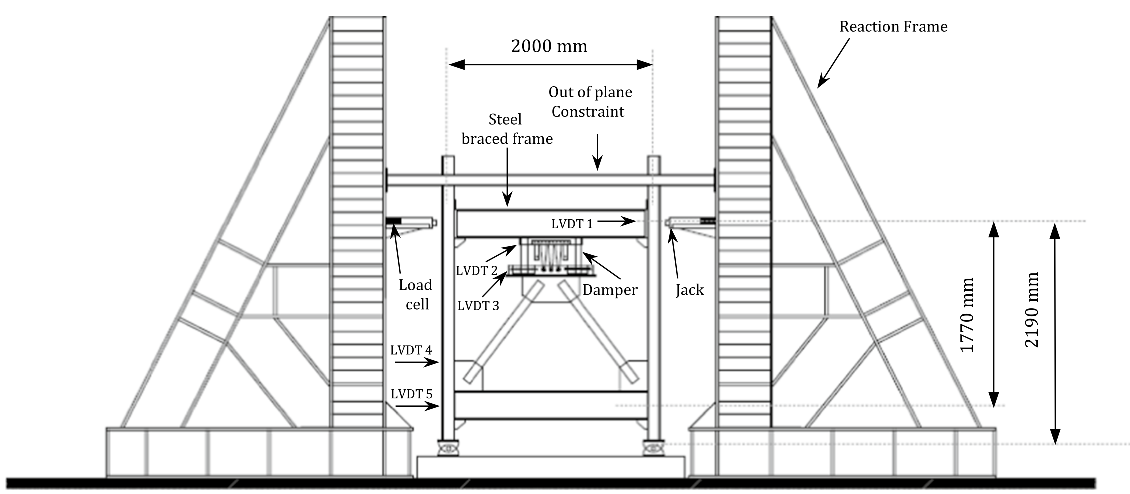


Fig. 2 Test setup for studied frames

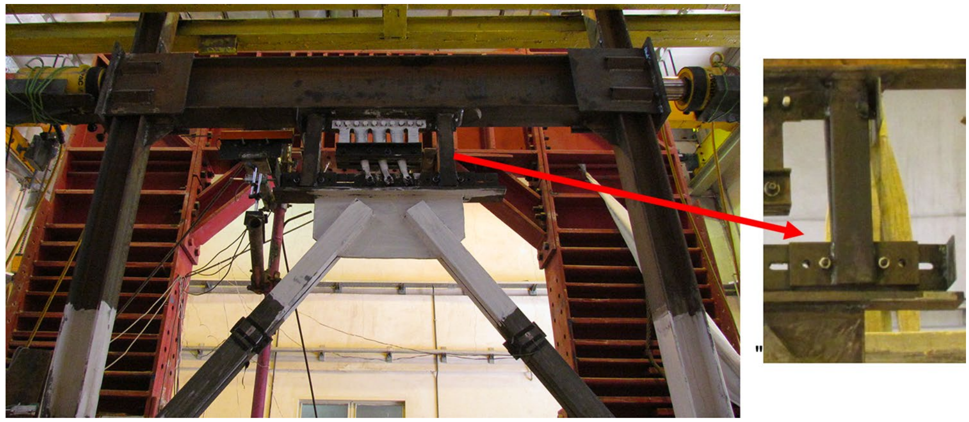


Fig. 3 A view of studied steel frame equipped with CTD

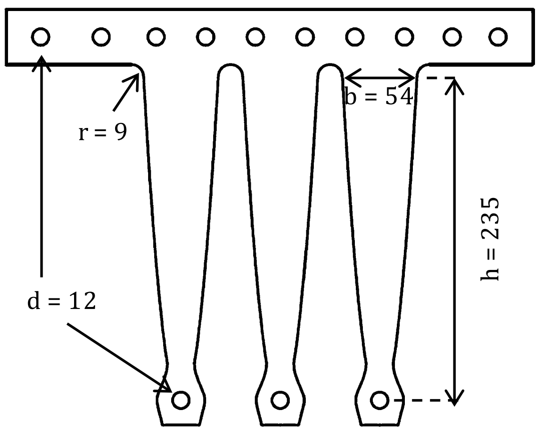


Fig. 4 Geometry of CTD3 (Garivani et al. 2016)

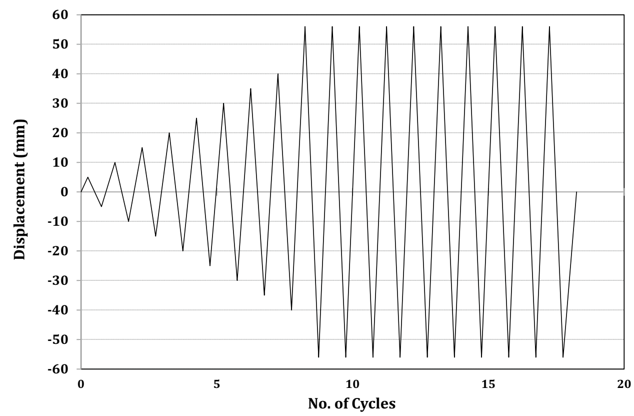


Fig. 5 Displacement history of Fr.1

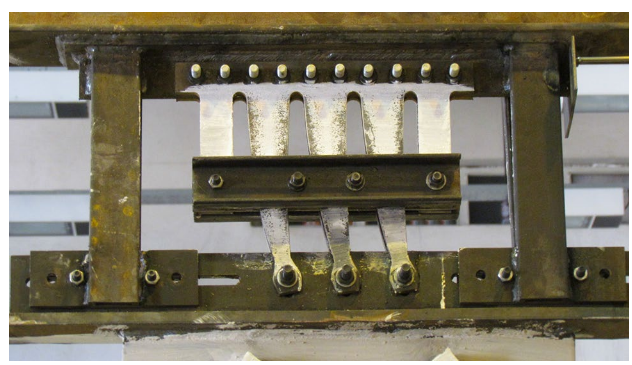


Fig. 6 Simultaneous yielding of outer fibers and its uniform growth along the damper teeth in Fr.1

two *T*-shaped restrainers were installed on both sides of the dampers. Due to details of their connections to the frame, these restrainers did not interfere with in-plane displacement of the damper. As shown, a brittle coating of lime was applied to the surface of CTDs to follow the evolution of yielding pattern in the teeth.

In order to guarantee the minimum intervention of frame action on the behavior of dampers, it was tried to detail the frame connection so that their moment bearing capacity is eliminated as much as possible. Therefore the gusset plates connecting the braces to the frame were only welded to the beam. Also the height of web clip angles connecting the beams to the columns were approximately equal to half of the beam web height.

3.1 Frame No. 1 (Fr.1)

The first frame (Fr.1) was equipped with a CTD same the CTD3 specimens studied by Garivani et al. (2016). Figure 4 shows the geometry of this specimen. The λ parameter defines the overall shape of this specimen was equal to $1.75\ mm^{0.5}$. The amplitude of lateral displacement cycles applied to the frame was gradually increased and finally 10 cycles of loading were applied with an amplitude corresponding to a story drift of 2.0% (Fig. 5). Figure 6 shows the deformed shape of the dampers, which was very similar to that observed during tests on the damper specimens.

In this frame, as the lateral displacement was increased, cracking started at the weld connecting the beam web angle to the column. It must be said that in this frame due to limited width of the column flange, the angle flange was quite short, limiting the flexibility of the beam to column connection. By increasing the frame lateral displacement, the cracks started to grow. Figure 7 shows this crack at the last stage of the experiment. The force–displacement curve of this frame is shown in Fig. 8. As seen in this figure, despite the mentioned crack growth during loading, the total stiffness of the frame was almost constant in consecutive cycles. It means that the stiffness of the simple steel frame is not comparable to that of the bracing system and consequently, the cracking at the mentioned weld connection has not affected the total stiffness of the frame equipped with CTD and so, it has a very stable hysteretic behavior.

In addition, during the test, no out-of plane deformation of bracing and damper systems was observed, which confirm the desirable performance of *T*-shaped restrainers. Also, due to special construction of frame connections, the hysteretic curves of the frame are quite similar to that of the individual dampers (Garivani et al. 2016). It must be noted, a CTD specimen identical to that use in Fr.1, had tolerate 20 full cycles of 40 mm displacement amplitude without any cracking or strength deterioration. The measurement of displacements at the top and bottom of CTD in Fr.1 produced the data about damper deformation, which is shown in Fig. 9. This figure shows that the maximum amplitude of this deformation has been 35 mm. The hysteretic curve of CTD2 specimen tested by Garivani et al. (2016) is also shown in Fig. 9. This specimen was similar to CTD3 and the one used in Fr.1, but had a different displacement history.

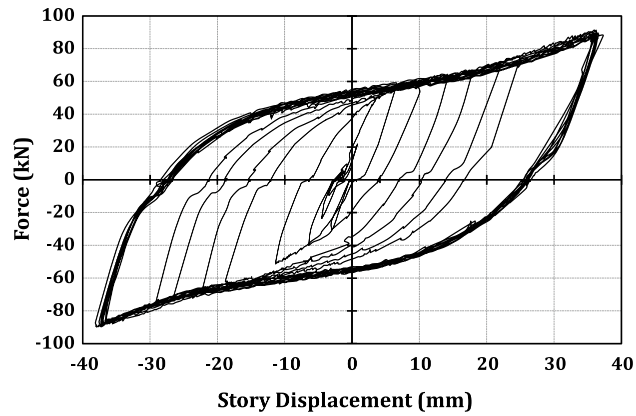


Fig. 8 Force–displacement curve of Fr.1

Although it had satisfactorily tolerated large displacements but in Fig. 9 experimental curve is shown only up to the level of deformations observed in Fr.1. The two curves show fairly good agreement confirming that in the frame specimen, the boundary conditions similar to that of the individual CTD specimen have been successfully reproduced.

3.2 Frame No. 2 (Fr.2)

As mentioned above, the test on Fr.1 showed that the details of connections and installation of the proposed damper were suitable for the intended purpose and the damper had similar behavior compared to that observed in tests carried out on individual specimens by Garivani et al. (2016). Therefore, in the second test, Fr.2 was constructed similar to Fr.1 in geometry and brace configuration, but a new geometry of

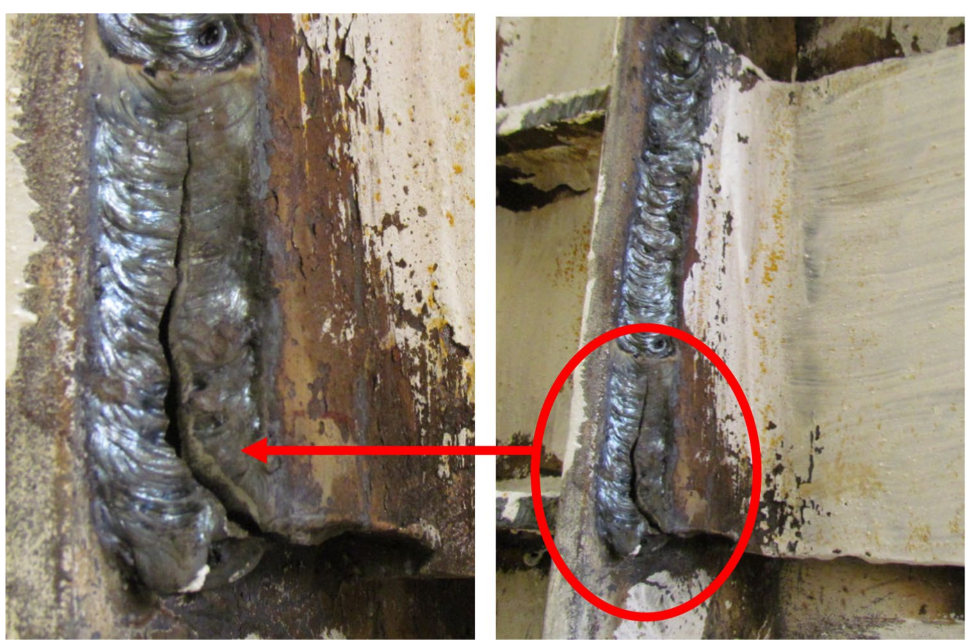


Fig. 7 Weld cracking in connecting area of web clip angles with columns in Fr.1

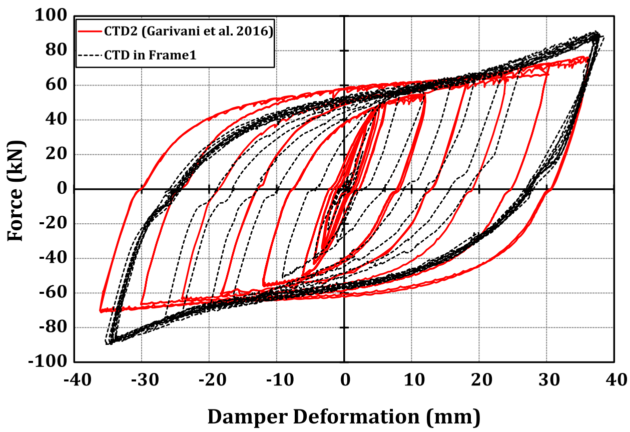


Fig. 9 Comparison of force–displacement curves of CTD2 tested by Garivani et al. (2016) with the damper used in Fr.1

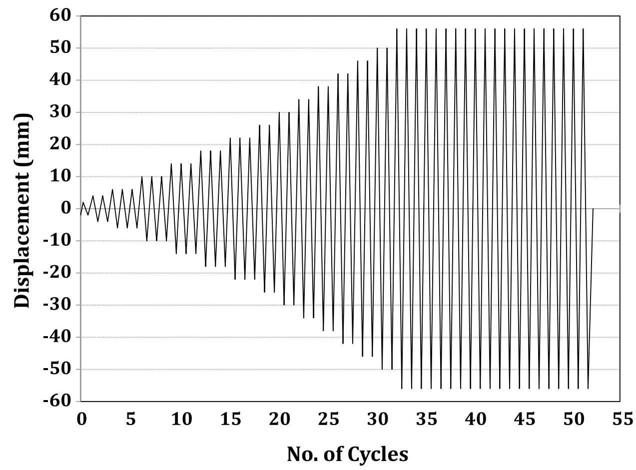


Fig. 11 Applied displacement history of Fr.2

Fig. 10 Using additional plate on the columns flanges to have more flexible web clip angles in Fr.2

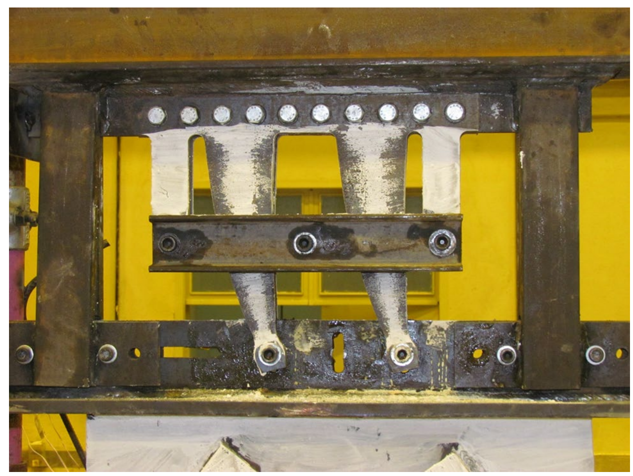


Fig. 12 Simultaneous yielding of outer fibers and its uniform growth along the damper teeth in Fr.2

CTD was used in this experiment to further verify the application of this type of damper. In addition, in order to have more flexible web clip angles in the bottom beam-to-column connection, the width of columns flange was increased by installing a plate on the columns flange (Fig. 10). This allowed using a large size angle producing more flexible connection between beam and column.

The CTD in this frame included two layers each consisting of two teeth. Dimension of each tooth as defined by parameters shown in Fig. 4 are as follows $t=10$ mm, $h=235$ mm and $b=77$ mm (which is obtained by $\lambda = 2.5$ mm$^{0.5}$). The displacement history applied to Fr.2 is shown in Fig. 11. As seen, in this experiment the displacement amplitude was different with Fr.1 so that it gradually increased to reach a value corresponding to the story drift of 2.0% and then, twenty cycles of loading were applied in this amplitude.

The behavior of CTD and frame in this test was similar to Fr.1, i.e. simultaneous yielding of outer fibers and its uniform growth along the teeth was observed (Fig. 12). In addition, the desirable performance of *T*-shaped out-of-plane restrainer was also observed during the test. The observation in this frame confirmed more flexible behavior of the beam to column connection in this frame as such no cracking in welds was observed before the drift ratio of 2%. Figure 13 shows the force–displacement curve of Fr.2. This frame has a very stable hysteresis curve and hence, behaves consistent with the design assumptions and goals.

Figure 14 shows the force–displacement curve of the damper used in Fr.2. The maximum displacement applied to

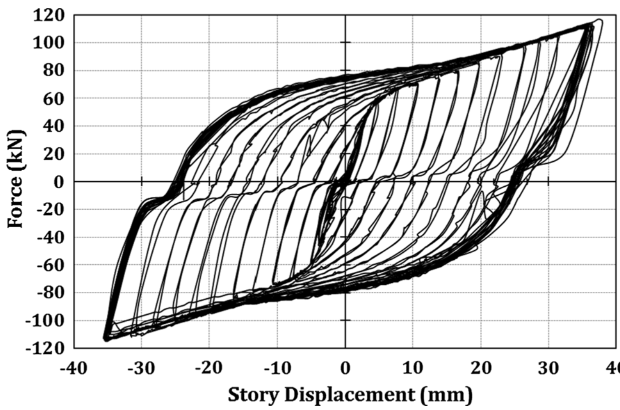


Fig. 13 Force–displacement curves of Fr.2

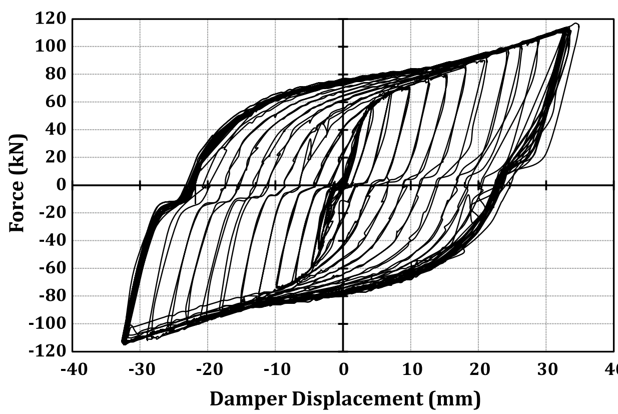


Fig. 14 Force–displacement curve of the CTD used in Fr.2

this damper is approximately 32 mm. On the other hand, considering the analytical equations presented by Garivani et al. (2016), the displacement corresponding to outer fiber yielding of CTD is equal to 1.3 mm. Therefore, this CTD has tolerated twenty fully reversed cycles at a displacement corresponding to almost 25 times of its yield displacement. It should be noted that at the end of this loading, there was no sign of any defect or cracking in the damper and it could still tolerate more cycles. Based on these results, it can be said that the comb-teeth dampers can be successfully used in structures as energy dissipation devices.

4 Numerical Study of Steel Frames Equipped with Comb-Teeth Damper

In order to study some more details of the behavior of frames equipped with CTD, the numerical model of Fr.2 was constructed and it was analyzed using the finite element method. Four node shell elements were used for modelling (Fig. 15). The material stress–strain data were extracted from a tensile coupon test reported by Garivani et al. (2016). According to the test results, the modulus of elasticity and initial yield stress of the steel are equal to 204 GPa and 274 MPa, respectively. The true stress–strain curve of steel material is shown in Fig. 16.

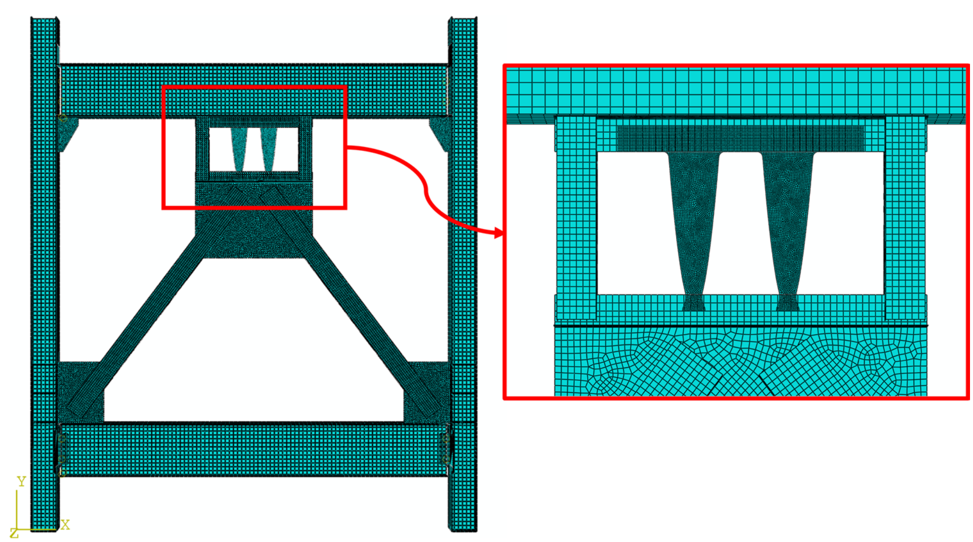
The hardening response was approximated using a combined nonlinear isotropic-kinematic hardening model. There, it is assumed that yield surface expansion is governed by the following form

$$\sigma = \sigma_0 + Q_\infty \left(1 - e^{-b\varepsilon_p}\right) \tag{2}$$

Q_∞ and b are the material parameters and ε_p is von Mises equivalent/effective plastic strain. σ_0 is the initial yield stress. Armstrong-Frederick, AF, formulation is used for the kinematic hardening rule, i.e.

$$\dot{\alpha} = C\dot{\varepsilon}_p \frac{1}{\sigma_0}(\sigma - \alpha) - \gamma\alpha\varepsilon_p \tag{3}$$

Fig. 15 FE model of tested Fr.2



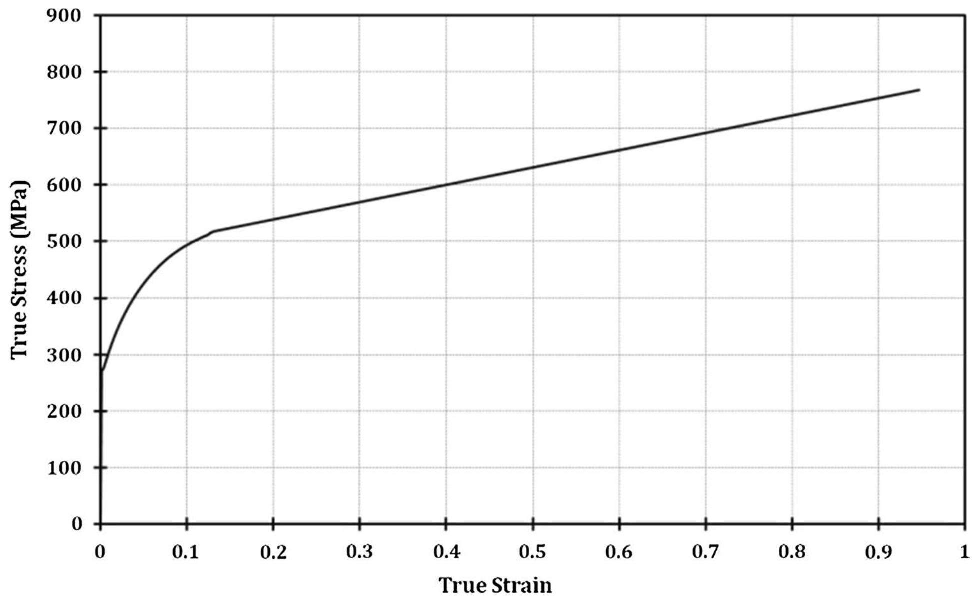


Fig. 16 True stress–strain curve of steel material

Table 1 Calibrated values of the model for the steel sample

σ_0 (MPa)	Q_∞	b
274	50	5

where σ and α denote stress and back stress tensors, respectively, and C and γ are material parameters. The dots on the quantities indicate their time derivatives. Table 1 presents the calibrated values of the model for the steel sample. They were calibrated in previous work of Garivani et al. (2016) so that the behaviors of individual CTD specimens obtained from numerical analyses and experimental program are in good agreement.

The FE model of the frame was analyzed under the displacement history applied during the tests. Figure 17 shows the results of FEA together with experimental one, which confirms fairly acceptable agreement.

Figure 18 shows the deformed shape of the frame and the von Mises stress distribution in selected parts of the frame at the final cycle of loading with maximum amplitude. Figure 18a confirms the desirable stress distribution in CTD. As expected most parts of the structure did not experience large stresses and they remained essentially elastic. The stress distribution in the upper beam is shown in Fig. 18b. As seen, largest stress in the beam occur in the web where the CTD is connected. Nevertheless, its magnitude is 140 MPa, which

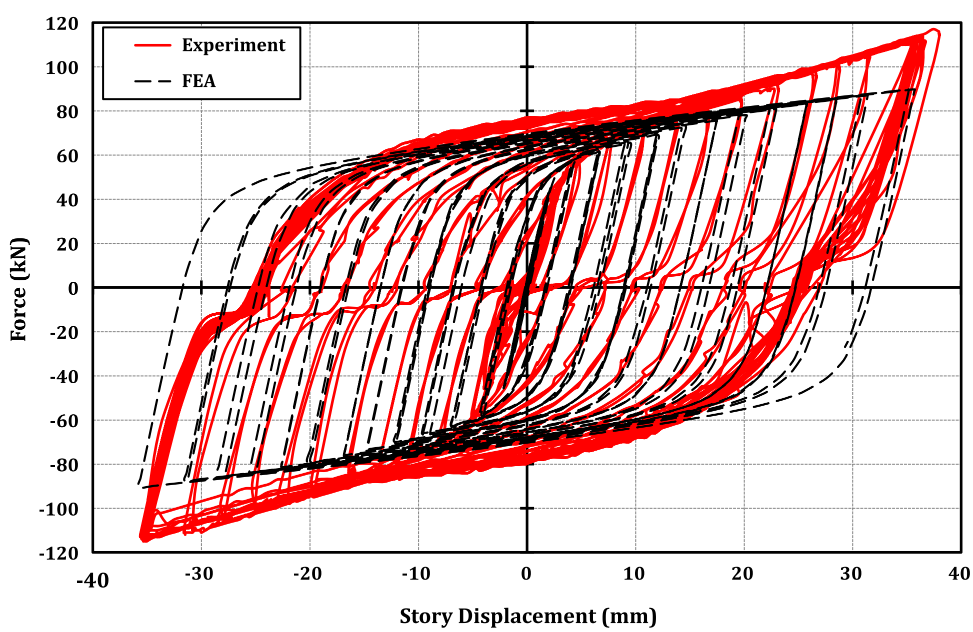


Fig. 17 Comparison of force–displacement curves obtained by experimental and numerical results

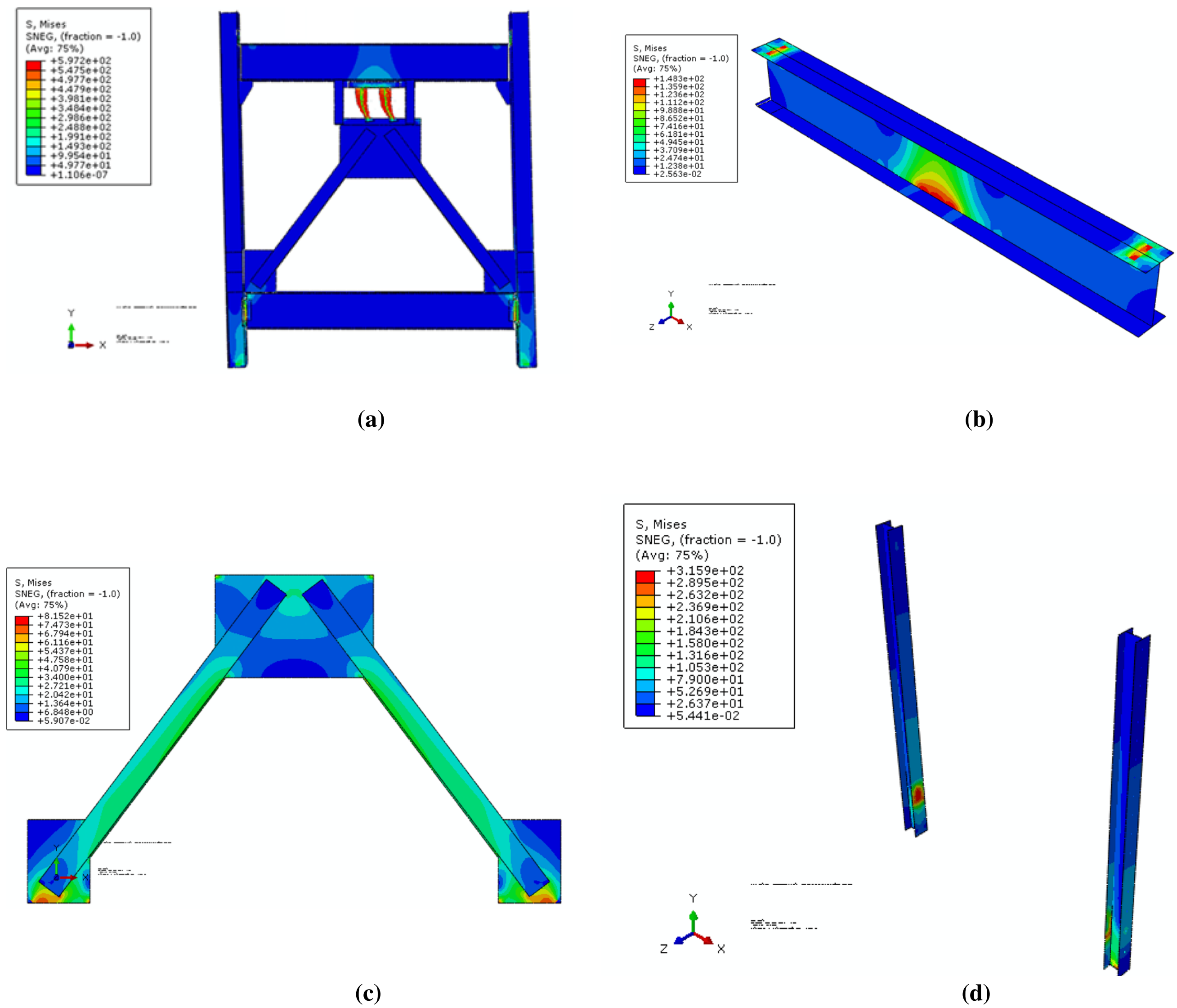


Fig. 18 FEM Model and von Mises stress distribution in frame elements. **a** Stress distribution in frame, **b** stress distribution in top beam, **c** stress distribution in braces and **d** stress distribution in columns

is below the yield limit. The deformed shape of the frame, beam and damper as well as the stress distribution confirms that by suitable proportioning of the frame elements, the buckling of braces are prevented and no unbalanced vertical force is applied to the beam. Also, due to special design of connections of the CTD, no axial load is produced in these elements either. Hence, the beam is basically subjected to some axial force and bending moment due to shear force and bending moments produced in the damper.

5 Investigating the Seismic Response of Framed Structures Equipped with CTD

In order to investigate the effects of using CTD on seismic behavior of framed structures, the finite element method was employed. For this purpose the frame used by Tsai et al. (1993) was selected as a reference. These researchers had investigated the seismic behavior of a reference frame with and without TADAS dampers. The frame setup used in their

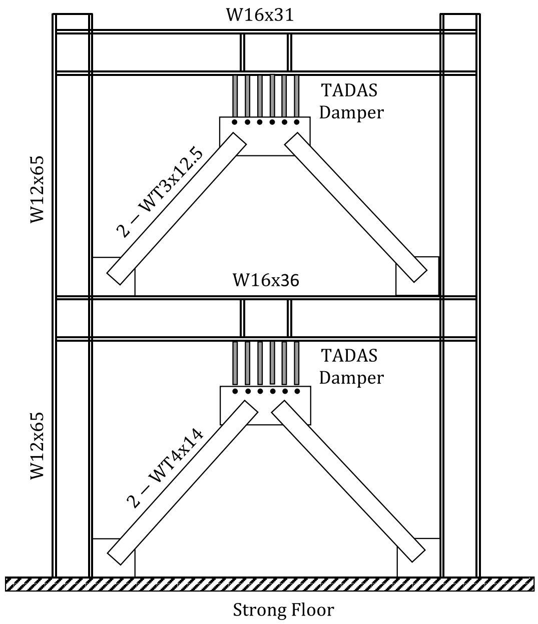


Fig. 19 Steel frame equipped with TADAS damper tested by Tsai et al. (1993)

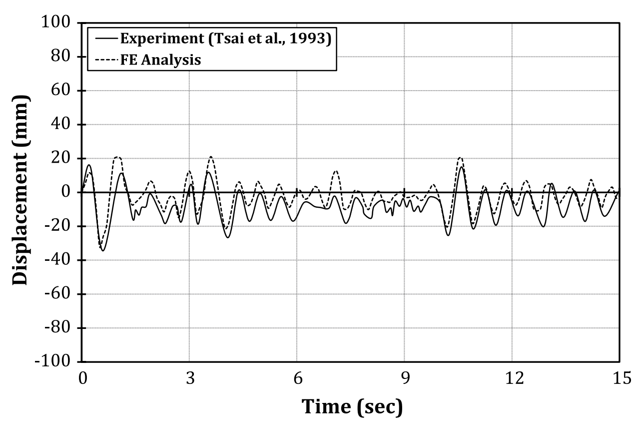


Fig. 20 First story displacement response of frame equipped with damper under ground motion with PGA = 312 cm/s^2

tests is shown in Fig. 19. The frame without any damper was tested subjected to 1940 El Centro earthquake record scaled to PGA of 50 cm/s^2 using a pseudo-dynamic technique. Subsequently the frame was equipped with a TADAS damper and then tested subjected to the same record with PGA equal to 50 cm/s^2 and 312 cm/s^2.

In the present study, the same frames were modeled and were subjected to the same ground motions. The model included frame elements for simulating the behavior of the beams, columns and bracings and nonlinear link elements for simulating the behavior of the damper. The behavior of

Table 2 Maximum displacement response of stories (experiment and FE analysis) (units: mm)

Method	Story displacement response	
	Story 1	Story 2
Experiment (Tsai et al. 1993)	34	82
FE analysis	32	77
Error	6.25%	6.49%

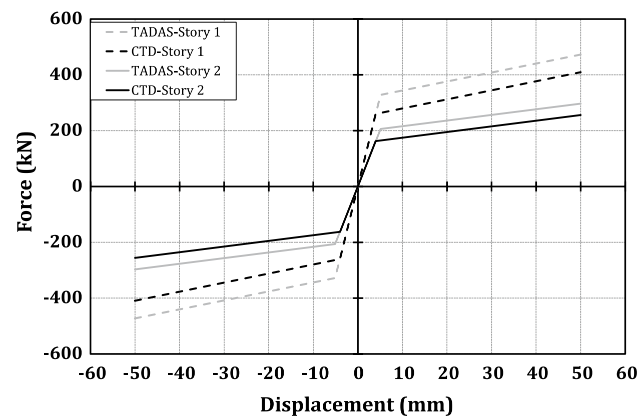


Fig. 21 Force–displacement curves of two type of damper in different stories

latter element was represented by a bilinear curve obtained from the available experimental results.

The results of FE models were very similar to those observed in experiments reported by Tsai et al. (1993). As an example, Fig. 20 shows a comparison between the experimental results reported by Tsai et al. (1993) and those of the FE model of this work for first story displacement response under mentioned ground motion with PGA = 312 cm/s^2. Maximum values of displacement response of the two stories are also listed in Table 2. They show a fairly good agreement confirming acceptable modelling of the frame and dampers behavior. It must be said that slight difference between the results might be due to the fact that the test was carried out using a pseudo-dynamic technique, while the FE model was subjected to a support excitation.

After verifying FE model, the same frame was equipped with comb-teeth dampers. The geometry of the CTD was assumed similar to those used in Fr.2 discussed in the previous sections. The number of CTD teeth were determined in a way that elastic stiffness of the dampers were similar to those of the TADAS dampers used in Tsai et al. (1993) work. The thickness of triangular plates used in the TADAS dampers was 36 mm and their height and base width were 325 mm and 178 mm, respectively. The number of plates in the first story was 8 and in the second one was 5. For the CTD, the geometry was as described in Sect. 3.2 and the number of teeth for the first and second stories were 16 and 10, respectively.

Figure 21 shows the force–displacement curves of the two types of damper in different stories. As shown in this figure, although the stiffness of dampers have been made the same, the yield strength of the CTD is slightly lower than TADAS due to their different mechanism. But this comparison implied that there was no need to redesign the braces of tested frame by Tsai et al. (1993).

The FE models of frames equipped with TADAS or CTD were then subjected to 20 different ground motion records. These ground motions were recorded on Soil Type C from real earthquakes and were also scaled to be compatible with a target design spectrum obtained based on ASCE-7 (2010). For constructing this target design spectrum, it was assumed that this frame is to be constructed in a location in California for which $S_s = 1.0\ g$ and $S_1 = 0.6\ g$. S_s and S_1 are spectral response acceleration parameters at short periods and a period of 1 s, respectively. The characteristics of these records are listed in Table 3. Figure 22 shows the average of scaled records response spectra compared with the design spectrum for the Maximum Considered Earthquake.

The maximum drifts experienced in the first and second stories of the frames, for each record, were obtained from the analyses and are shown in Figs. 23 and 24, respectively. The average of response from different records are also shown in these figures, which indicate that they are pretty similar and for the frame equipped with CTD, in both stories average responses are only about 10% greater than frame equipped with TADAS.

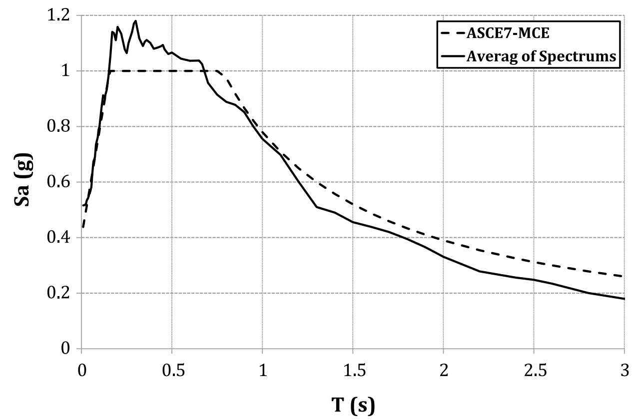
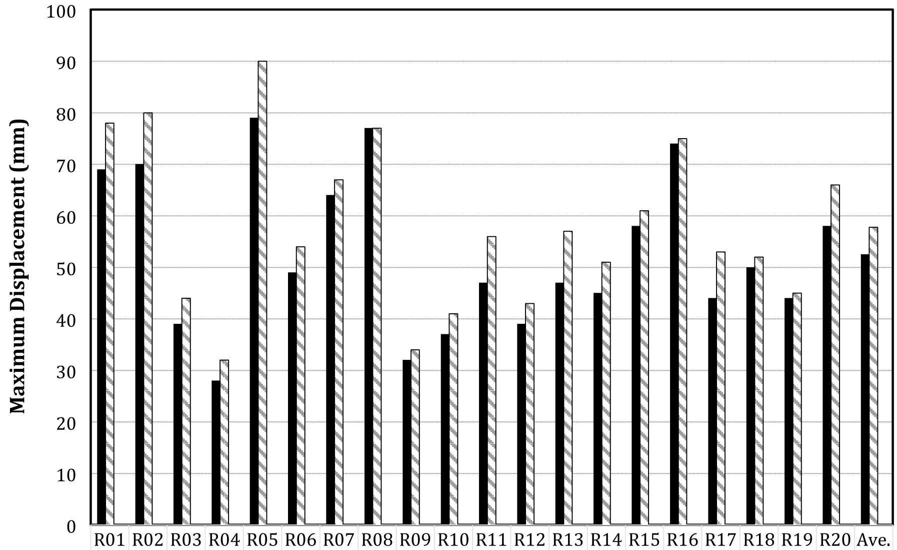


Fig. 22 Average elastic response spectrum and design spectrum

Despite fairly similar response of the two types of frames, one could note that the volume/weight of steel used in TADAS is 4.2 times the used in CTD. Also, another parameter which affects the economy of the design is the cutting area required for constructing the dampers from steel plates. A comparison of this parameter also shows that the cutting area required for TADAS is 2.5 times that of CTD. The above results confirm that CTD dampers can offer the same efficiency observed in TADAS dampers with lower manufacturing costs.

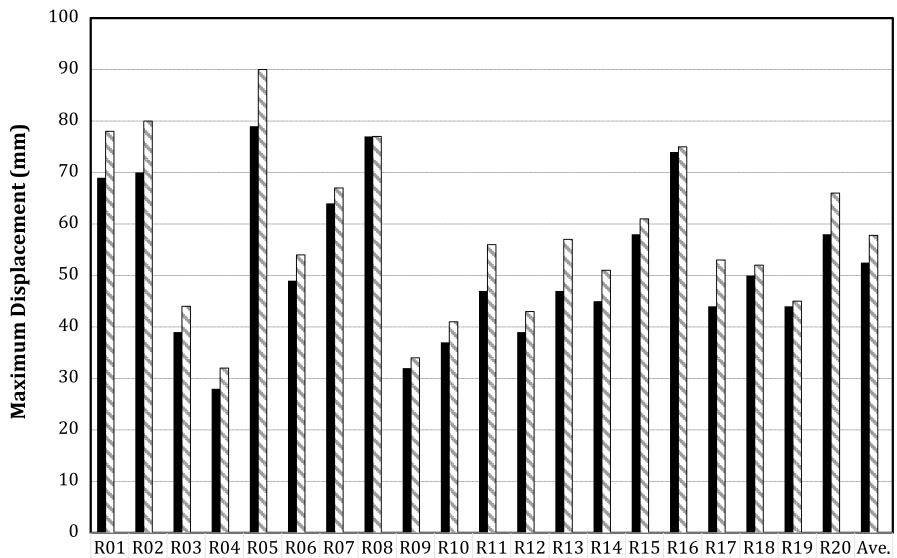
Table 3 Characteristics of real records used for analysis

Record name	Scale factor	5–95% Dur. (s)	Earthquake name	Year	Station name	Magnitude
R01	6.94	29.5	Kern County	1952	Pasadena - CIT Athenaeum	7.36
R02	3.87	33.6	Kern County	1952	Santa Barbara Courthouse	7.36
R03	3.01	30.3	Kern County	1952	Taft Lincoln School	7.36
R04	9.12	29	Parkfield	1966	Cholame - Shandon Array #12	6.19
R05	42.42	37.4	Borrego Mtn	1968	Pasadena - CIT Athenaeum	6.63
R06	12.91	28	Borrego Mtn	1968	San Onofre - So Cal Edison	6.63
R07	2.25	16.8	San Fernando	1971	Castaic - Old Ridge Route	6.61
R08	4.44	18.4	San Fernando	1971	Lake Hughes #1	6.61
R09	56.09	23.4	San Fernando	1971	Maricopa Array #3	6.61
R10	4.00	14.5	San Fernando	1971	Pasadena - CIT Athenaeum	6.61
R11	13.27	48.2	San Fernando	1971	San Juan Capistrano	6.61
R12	27.97	35.5	San Fernando	1971	San Onofre - So Cal Edison	6.61
R13	21.96	7.1	Oroville-02	1975	Up and Down Cafe (OR1)	4.79
R14	15.80	7	Santa Barbara	1978	Cachuma Dam Toe	5.92
R15	3.21	7.5	Santa Barbara	1978	Santa Barbara Courthouse	5.92
R16	23.08	14.9	Norcia (Italy)	1979	Bevagna	5.9
R17	2.55	36.4	Imperial Valley-06	1979	Cerro Prieto	6.53
R18	6.80	14.2	Livermore-01	1980	San Ramon - Eastman Kodak	5.8
R19	9.40	27.1	Livermore-01	1980	San Ramon Fire Station	5.8
R20	14.51	8.7	Livermore-02	1980	Fremont - Mission San Jose	5.42



	R01	R02	R03	R04	R05	R06	R07	R08	R09	R10	R11	R12	R13	R14	R15	R16	R17	R18	R19	R20	Ave.
TADAS Dampers	69	70	39	28	79	49	64	77	32	37	47	39	47	45	58	74	44	50	44	58	53
CTDs	78	80	44	32	90	54	67	77	34	41	56	43	57	51	61	75	53	52	45	66	58
CTD /TADAS	1.13	1.14	1.13	1.14	1.14	1.10	1.05	1.00	1.06	1.11	1.19	1.10	1.21	1.13	1.05	1.01	1.20	1.04	1.02	1.14	1.10

Fig. 23 Maximum relative displacement response story 1



	R01	R02	R03	R04	R05	R06	R07	R08	R09	R10	R11	R12	R13	R14	R15	R16	R17	R18	R19	R20	Ave.
TADAS Dampers	69	70	39	28	79	49	64	77	32	37	47	39	47	45	58	74	44	50	44	58	53
CTDs	78	80	44	32	90	54	67	77	34	41	56	43	57	51	61	75	53	52	45	66	58
CTD /TADAS	1.13	1.14	1.13	1.14	1.14	1.10	1.05	1.00	1.06	1.11	1.19	1.10	1.21	1.13	1.05	1.01	1.20	1.04	1.02	1.14	1.10

Fig. 24 Maximum relative displacement response story 2

6 Conclusions

In this paper, the behavior of steel frames equipped with comb teeth dampers (CTD) was studied experimentally and numerically. The results of cyclic test on two frames equipped with CTD showed that this type of damper can successfully be used in real structures and they can behave as foreseen in design and observed in individual specimens and can dissipate significant amount of energy.

FE analysis of the experimental frame confirmed that by suitable proportioning of frame members and using buckling restraining element, buckling of brace elements are prevented and all elements of frame except for damper remain effectively elastic. This makes CTD a replaceable energy dissipating device. Nonlinear time history analysis of a reference frame subjected to a set of ground motions also showed that CTD can be as effective as TADAS in reducing seismic demand of structures, while offering a superior economy.

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