Bracing Design in Dual Systems for Earthquake Resistance

T. Sophocleous & M.C. Phocas
Department of Architecture, Faculty of Engineering, University of Cyprus, Cyprus

SUMMARY:
Structural control through energy dissipation systems has been increasingly implemented internationally in the last years and has proven to be a most promising strategy for earthquake safety of the structures. The present paper examines an alternative control system development for achieving dynamic structural adaptability. The prototype control system consists of a hysteretic energy dissipation device of triangular shaped steel plates and a tension only bracing mechanism with a kinetic closed circuit in two different configurations. The bracing-damper mechanism does not practically affect the initial stiffness of the system, i.e. the concept relies on two completely "separate" systems: the primary frame responsible for the vertical- and wind loads and the control system, for the earthquake loads. The predominant parameters that characterize the system's seismic behaviour are verified in respect to predefined mechanical properties of the control elements under the action of ten selected earthquake excitations of the Greek-Mediterranean region.

Keywords: Dual Systems, Hysteretic Dampers, Passive Control

1. INTRODUCTION

The design of frame structures with added control members for earthquake resistance refers primarily to the need for the primary systems to exhibit a linear elastic behaviour under seismic actions. A reduction of the energy dissipation demand on primary structural systems was successfully aimed at by a number of researchers (Constantinou and Symans 1993; Symans *et al.* 2008). Passive metallic yielding, friction, viscoelastic and viscous damping devices may be added to frame structures to dissipate input energy during an earthquake and to substantially reduce or eliminate damage to the gravity-load-resisting frames (Di Sarno and Elnashai 2005). ADAS and TADAS are known examples, available for both, new seismic resistant designs and retrofit of frame structures (Tsai et al. 1993; Dargush and Soong 1995). In principle, steel plate dampers are introduced in moment resisting frames with chevron bracings of large hollow section diagonals. The bracing members are typically connected to the beams and the columns with gusset plates through welting or bolting. In experimental prototype tests conducted the devices showed a stable hysteretic behaviour under a number of loading cycles (Tsai et al. 1993; Symans *et al.* 2008). Nevertheless the bracing components increase the overall stiffness of the system, as they consist of steel members stressed in compression, tension and bending. Before yielding of the integrated damper's plates, such stiff bracings may reduce inter-story displacements, while also developing high accelerations. In addition the application of the members under compression leads to a relatively inefficient behaviour under cyclic loading; in every half-loading cycle the compression diagonal buckles and it therefore cannot participate in the energy dissipation process.

Slender bracing members have found up to date limited applications for the integration of dampers in frame structures (Di Sarno and Elnashai 2005). A reason for this is their tendency of becoming slack under tension yielding and compression buckling. In addition sudden increases of the tensile forces in the slender braces create detrimental impact loadings on the connections and the other structural members. The amplification of the static response of the structure due to such impact loading may be accounted for by a respective impact factor (Tremblay and Filiatrault 1996). On the other side the application of a tension bracing-mechanism for earthquake resistance purposes seems to be a promising alternative as regards avoidance of stiffness interaction with the primary system, as well as achievement of simplicity and aesthetic qualities of the structure in broader architectural context. The implementation of tension-only bracings with damping devices in frame structures may only be realized through the development of suitable bracing-damper configurations, whereas all bracing members contribute during the entire load duration to the operation of the integrated damper. In this way optimization of the control system's operation principles for earthquake structural resistance may be achieved.

A control mechanism that enables the contribution of all structural members in the energy dissipation process is the Pall-Marsh friction mechanism with slender cross braces, as configured in (Filiautrault and Cherry 1988). The rectangular damper deforms into a parallelogram, dissipating energy at the bolted joints through sliding friction. With the completion of a loading cycle, the resulting areas of the hysteresis loops are identical for both braces. An alternative friction mechanism configuration with cross braces has also been proposed by Wu *et al.* (2005). An implementation of chevron cable members with a friction damper consisting of three rotating plates and circular friction pad discs placed in between is described in (Mualla and Belev 2002).

Proposed control systems consisting of hysteretic dampers and slender bracing members are based in their operation on relative displacements between the tension-only members. Hysteresis is achieved through optimization of the integrated hysteretic dampers plates' section. The cross braces with an articulated quadrilateral with steel dissipaters as proposed in (Di Sarno and Elnashai 2005; Renzi *et al.* 2007), work only in tension, whereas energy dissipation develops through elasto-plastic flexure of the steel plates with varying depth. A similar cross cable bracing configuration has been proposed by Kurata, DesRoches and Leon (2008), with a central energy dissipater of two steel plates that are interconnected through a rotational spring and eight elastic cables. Under seismic excitation four cables in tension rotate the steel plates in opposite directions. The remaining cables connecting across the shortened diagonal are stressed elastically in compression and do not become slack, when the loading direction changes, due to permanent rotations of the steel plates.

The development of Adaptable Dual Control Structures, ADCS, is originally presented in (Phocas and Pocanschi 2003). In principle ADCS consist of a cable bracing with closed circuit and a hysteretic damper of steel plates. During strong ground motions relative displacements between the bracing and the frame member interconnected through the hysteretic damper yield to the damper's own deformations and energy dissipation. ADCS is only responsible for the earthquake forces enabling in all cases the elastic response of the primary system.

ADCS introduce a prototype connections design for the bracing members, based on rotating discs. The connection principle may be applied in different bracing configurations that share common features in respect to the kinetic model and the control behaviour of the system (Sophocleous and Phocas 2009a). Furthermore, the hysteretic damper applied in ADCS, may follow the section principles of hysteretic dampers consisting of X- or triangular-shaped steel plates for achieving uniform deformation curvatures over the sections' height, as applied in ADAS and TADAS (Xia and Hanson 1992; Aiken *et al.* 1993; Tsai *et al.* 1993). The present analysis refers to two particular ADCS-configurations, a portal bracing, ADCS1 and a portal- and a chevron bracing, ADCS2 (Sophocleous and Phocas 2009b; Sophocleous and Phocas 2010). Each configuration provides a differentiated seismic performance, but also an alternative structural form that can be applied within the broader architectural context of the building. Optimal system parameters for each configuration of the damper-bracing mechanism have been derived under three international strong earthquake motions, as regards the energy dissipation behaviour of the systems. In the present paper the systems are verified in their response under ten earthquake motions of the Greek Mediterranean region.

2. DESIGN CONFIGURATION

The bracing members of the proposed control systems are connected at the bottom of the columns and are free to move at their connecting joints to rotating mechanical discs, Fig. 2.1. In both configurations a hysteretic damper is placed between the beam and the horizontal bracing member. In ADCS2 a pair of chevron braces is additionally connected to the portal bracing, to a middle eccentric disc connected at the lower horizontal connecting plate of the damper. The position of the hysteretic damper within the system was selected for enabling maximum performance in utilizing the maximum relative displacement between its connection points. The hysteretic damper consists of a suitable number of identical triangular-shaped, mild steel plates positioned in parallel and welded on two horizontal plates. The dampers plates’ section lines reach their maximum yielding potential at the same time under the developed shear forces.

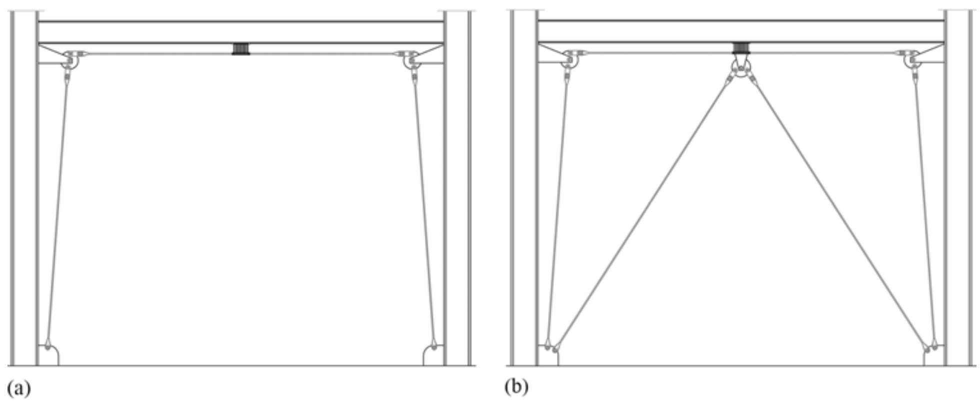

Figure 2.1. Adaptable Dual Control System with tension-only bracing-damper mechanism: (a) Portal bracing configuration, ADCS1; (b) Portal- and chevron bracing configuration, ADCS2

ADCS kinetic mechanism is activated during the dynamic excitation by the horizontally induced motion at the base of the structure. In every half-loading cycle the respective displacement of the primary frame is followed by the bracing members through rotations of the eccentric discs. Since the bracing members form a closed circuit, ideally the reactions on the primary frame at the end of each cycle of movement are neutralized and the members remain under tension, Fig. 2.2. The optimization of the bracing-damper mechanism involves tuning between the stiffness, the yield force and the shear deformations of the hysteretic damper for maximum energy dissipation. In addition the maximum base shear and relative displacements of the controlled system should be held in bounds with the respective responses of the primary system.

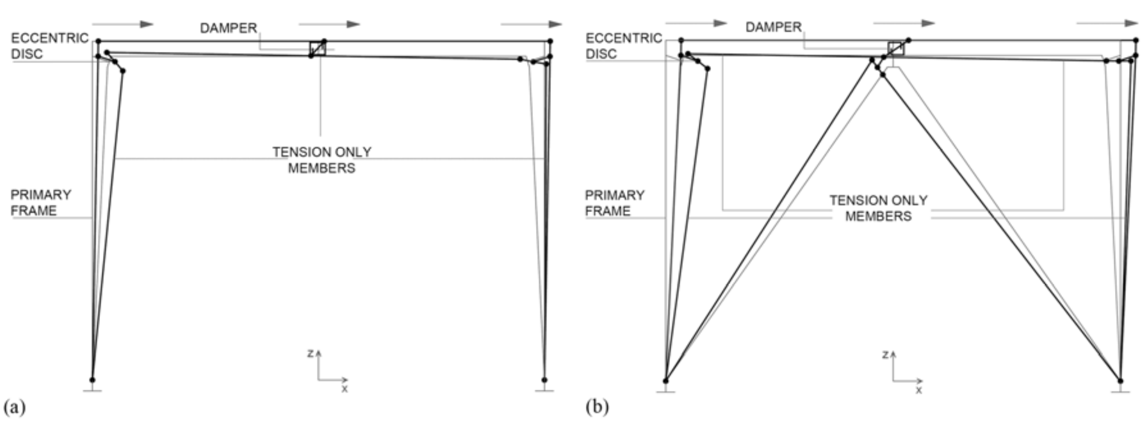

Figure 2.2. Kinetic systems’ models: (a) ADCS1; (b) ADCS2

3. SYSTEM MODELS

The finite element analysis of ADCS relies on a simplified model, whereas non-linearity is only addressed to the hysteretic damper. ADCS dynamic behaviour was examined with the software program SAP2000. A typical geometry was assigned for the ideal SDOF-model of a moment resisting steel frame of 6.0 m long beam and 4.5 m high column members. The preliminary analysis was completed for a number of combinations of characteristic design variables. IPBv500 sections were assigned for the columns and IPBL550, for the beam (S235, E= 2.1x10^4 kN/cm^2, ρ= 78.5 kN/m^3). The dimensioning of the members was based on Eurocode 3, having assumed a static vertical load of 1200 kN, a horizontal wind load of 15 kN and 25 % of the vertical load as static equivalent seismic load. The primary frame's fundamental period results to $T = 0.34$ s and its stiffness to $k = 41717.37$ kN/m.

A diaphragm constraint at the roof level is required for the planar controlled systems. Especially, the portal bracing in ADCS1 requires a rigid diaphragm in the perpendicular plane direction at the primary beam level, due to its high sensitivity in respect to out of plane deformations that influence negatively the respective energy dissipation during the seismic excitation. The bracing members in this particular configuration consist of steel rods with constant diameter of d_c = 20 mm. In ADCS2, cables of the same diameter have been applied for all bracing members (E= 1.6x10^4 kN/cm^2, f_e = 140 kN/cm^2). The effective bracings' stiffness values, k_b, for the two configurations account to 27.78 and 2895.34 kN/m respectively. For the respective estimation, the stiffness of the vertical bracing's member was decisive in ADCS1, whereas in ADCS2, the respective values of the vertical and diagonal braces connected in parallel. In ADCS1 the bracing members were modelled as frame objects with zero compression limits and in ADCS2 the cables were additionally assigned a suitable pretension stress that would keep the members straight and taut when they are deformed. The rotating discs were modelled as composition of three short frame members, each assigned with large stiffness values to represent the real property of a shaft.

3.1. Mechanical Properties of Hysteretic Damper

Hysteretic dampers may exhibit elasto-plastic or rigid-plastic behaviour. The damper used in ADCS was modelled as a non-linear link element. The optimization method implies the selection of the maximum deformation and yield strength for the selected damper plates under the indicated hysteretic behaviour based on the Bouc-Wen plasticity model of hysteresis. Two characteristic parameters control decisively the system's response: the damper's initial elastic stiffness, k_d, and its yield force, P_y. These are given through the following Equations:

$$k_d = \frac{nEbt^3}{6h^3} \tag{3.1}$$

$$P_y = \frac{nf_y bt^2}{6h} \tag{3.2}$$

where h is the steel plate's height, b is the -width (fixed to the beam), t is the -thickness, n is the number of steel plates and f_y is the yield stress (S235, E= 2.1x10^4 kN/cm^2, f_y = 24 kN/cm^2, ρ= 78.5 kN/m^3). A design parameter, defined as Damper Ratio, DR, that describes ADCS response as a function of the damper's stiffness and -yield force may be introduced, as follows:

$$DR = \frac{k_d}{P_y} \tag{3.3}$$

By substituting Eqns 3.1 and 3.2 in Eqn. 3.3, DR may be written in the following form:

$$DR = \frac{Et}{f_y h^2} \tag{3.4}$$

ADCS performance index for structural safety has been defined as Effective Energy Deformation Index, EEDI that physically represents the amount of input seismic energy dissipated by the hysteretic device in the entire seismic time duration. ADCS have been subjected to international strong earthquake records characterised by peak ground acceleration values of 0.810g, 0.604g and 0.348g. The optimal design parameters of the systems have been derived based on numerical analyses results for 366 DR-combinations for ADCS1 and 397 for ADCS2. The selected geometry of the damper's steel plates for highest possible energy dissipation and limitation of the maximum base shear and relative displacements of the systems accounts to n= 2, t= 2.8 cm, h= 25 cm and b= 10 cm (damper: 2282510; DR= 392 1/m, k_d = 9835 kN/m, P_y = 25.09 kN) for ADCS1, and n= 6, t= 1.6 cm, h= 35 cm and b= 5 cm (damper: 616355; DR= 114.3 1/m, k_d = 1003.1 kN/m, P_y = 8.78 kN) for ADCS2.

3.2. Input Seismic Records

The selected designs of the adaptable dual control systems in the two configurations are verified based on ten seismic input records from the Mediterranean earthquake prone area. Both, the peak ground acceleration and the frequency content are different for each of the selected records, while the time duration varies in the range between 13.91 and 46.01 s, Fig. 3.1. The dynamic response verification studies aim at providing both validation and reliability of ADCS potential for ensuring earthquake safety of the primary system.

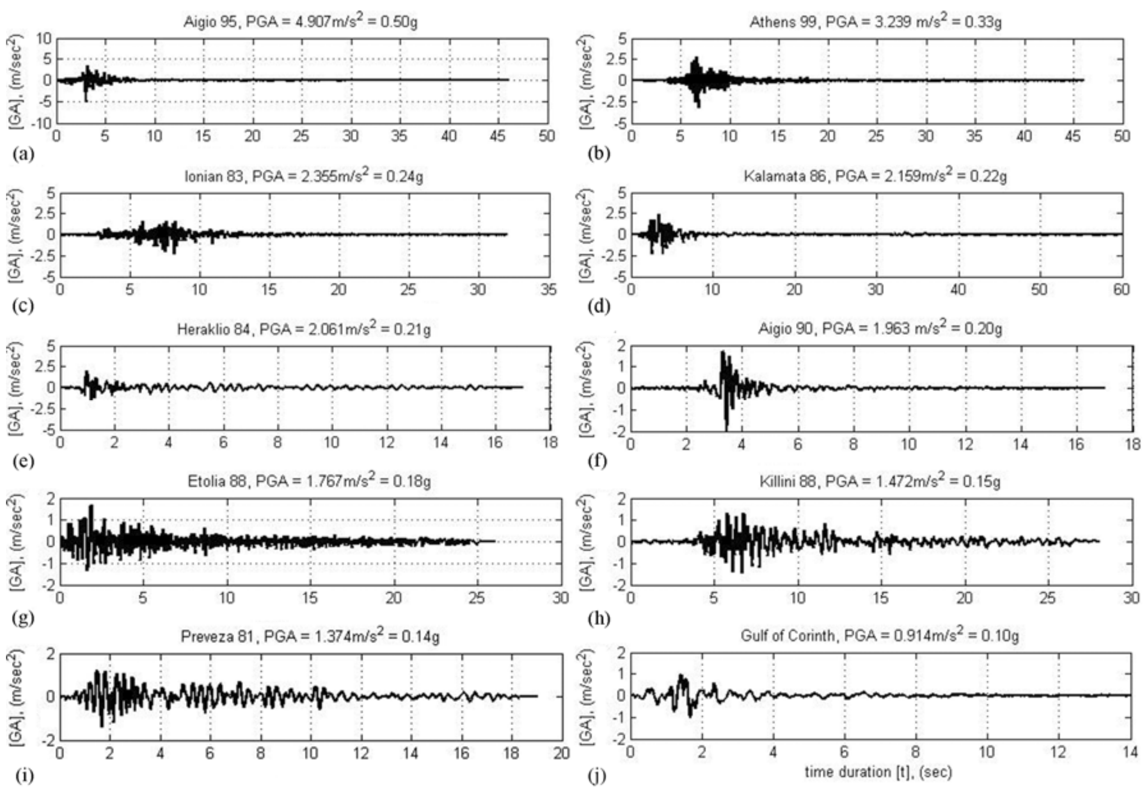

Figure 3.1. Greek Mediterranean seismic input records

4. SYSTEMS DYNAMIC RESPONSE

4.1. Natural Period

The controlled system's period is associated to the behaviour of the system in its linear elastic range, depending on the primary frame's- k and the damper's stiffness k_d that are linked in parallel, while remaining independent of P_y, which represents the nonlinear behaviour of the damper (Nakashima, Saburi and Tsuji 1996). Compared to the primary frame's fundamental period of $T = 0.34$ s, ADCS1 period decreases to the range of $0.278 < T < 0.288$ s, whereas ADCS2 period decreases to the range of $0.268 < T < 0.272$ s according to the respective damper's stiffness.

4.2. Energy Dissipation

The capability of ADCS1 configuration to enhance energy dissipation under the action of the ten selected seismic records was verified for the selected damper's steel plates. EEDI reaches 73.98 % in seismic case 1, 88.36 % in case 2, 27.90 % in case 3, 79.00 % in case 4, 28.82 % in case 5, 56.82 % in case 6, 70.20 % in case 7, 52.45 % in case 8, 67.23 % in case 9, whereas practically no energy dissipation is succeeded in case 10, Fig. 4.1. This means that in 30 % of the cases the system performs unsatisfactorily, i.e. seismic cases 2, 4 and 10, whereas in 20 % of the cases the system's performance is on average, i.e. seismic cases 6 and 8, and in half of the cases the system performs successfully with an energy dissipation of over 67.00 % of the input energy. On average, EEDI for all ten earthquake excitations accounts to 54.48 %. ADCS2 respective EEDI values are as follows: 92.36 % in seismic case 1, 88.96 % in case 2, 92.04 % in case 3, 95.39 % in case 4, 81.57 % in case 5, 76.41 % in case 6, 91.52 % in case 7, 79.50 % in case 8, 70.03 % in case 9 and 86.82 % in case 10. In all the events ADCS2 dissipates on average 85.46 % of the input energy, Fig. 4.2.

4.3. Base Shear

The maximum base shear responses of the controlled systems obtained in the ten seismic cases are compared to the primary frame's respective responses in Table 4.1. As far as ADCS1 is concerned, in 10 % of the cases (seismic case 7) the maximum base shear is increased by 59.37 %, whereas in all other cases there is a considerable respective decrease, as follows: 14.00 % in case 1, 12.00 % in case 2, 5.80 % in case 3, 4.69 % in case 4, 57.97 % in case 5, 13.86 % in case 6, 73.91 % in case 8, 32.68 % in case 9 and 3.59 % in case 10. On average ADCS1 causes 15.91 % decrease of the controlled system's base shear. On the other hand, in 30 % of the cases examined the maximum base shear of ADCS2 increases slightly: By 29.00 % in seismic case 3, 18.00 % in case 7 and by 0.50 % in case 10, Table 4.1. The respective response decreases by 11.52 % in case 1, 7.50 % in case 2, 6.70 % in case 4, 27.39 % in case 5, and by 11.35 % in case 6. On average ADCS2 causes 1.70 % decrease of the controlled systems base shear.

Table 4.1. Primary frame's, ADCS1 and ADCS2 maximum base shear BS

Seismic case	Max. base shear [kN]		
	Primary frame	ADCS1	ADCS2
1	1577.00	1355.27	1395.31
2	2048.00	1798.69	1894.39
3	500.30	471.28	645.57
4	1278.00	1218.54	1192.3
5	1738.00	730.50	1261.95
6	1361.00	1172.43	1206.59
7	882.90	1407.06	1042.11
8	2516.00	656.47	1389.42
9	2445.00	1646.02	2094.37
10	519.10	500.47	521.68

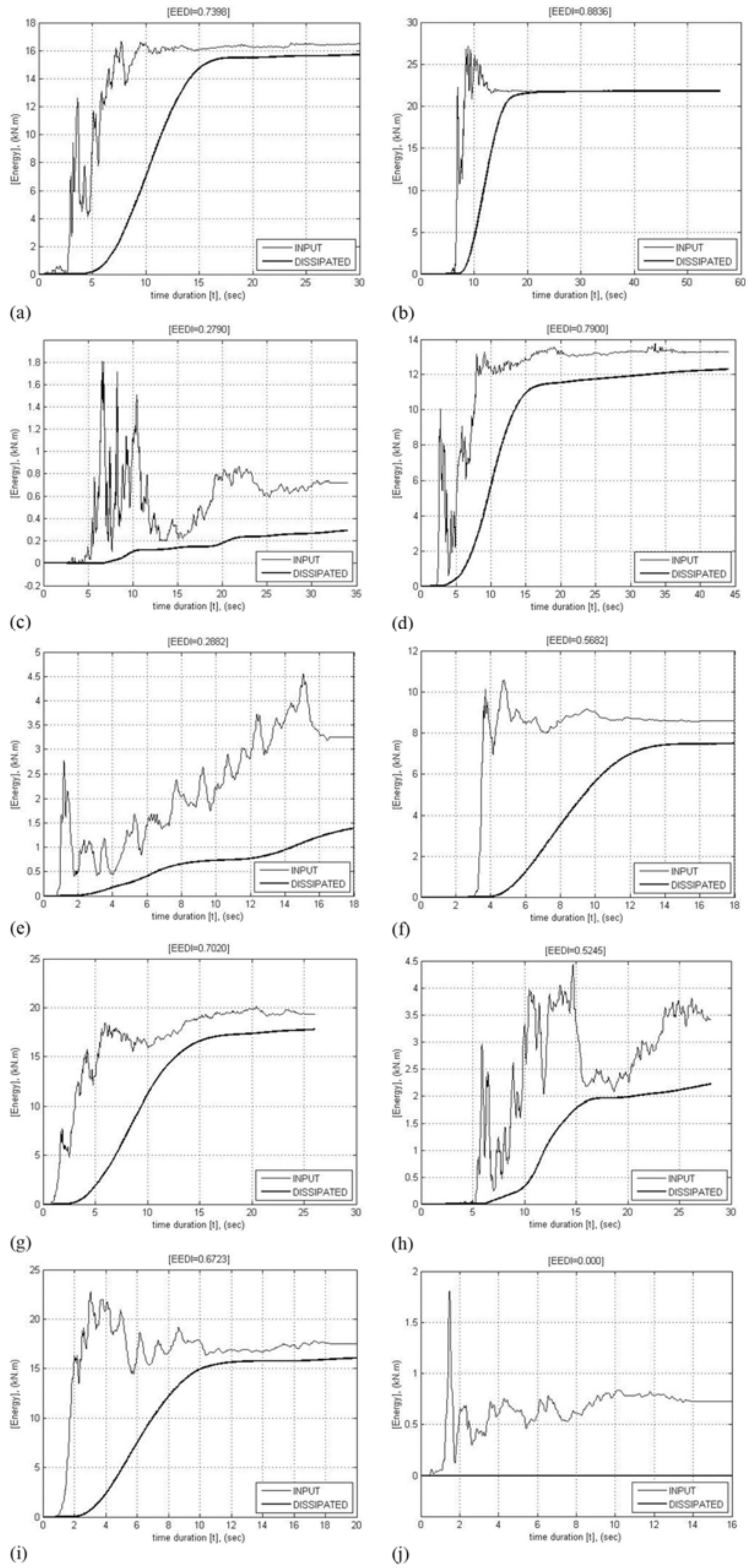

Figure 4.1. ADCS1 hysteretic damper's energy dissipation and force-deformation behaviour (damper: 2282510): (a-j) seismic case 1-10

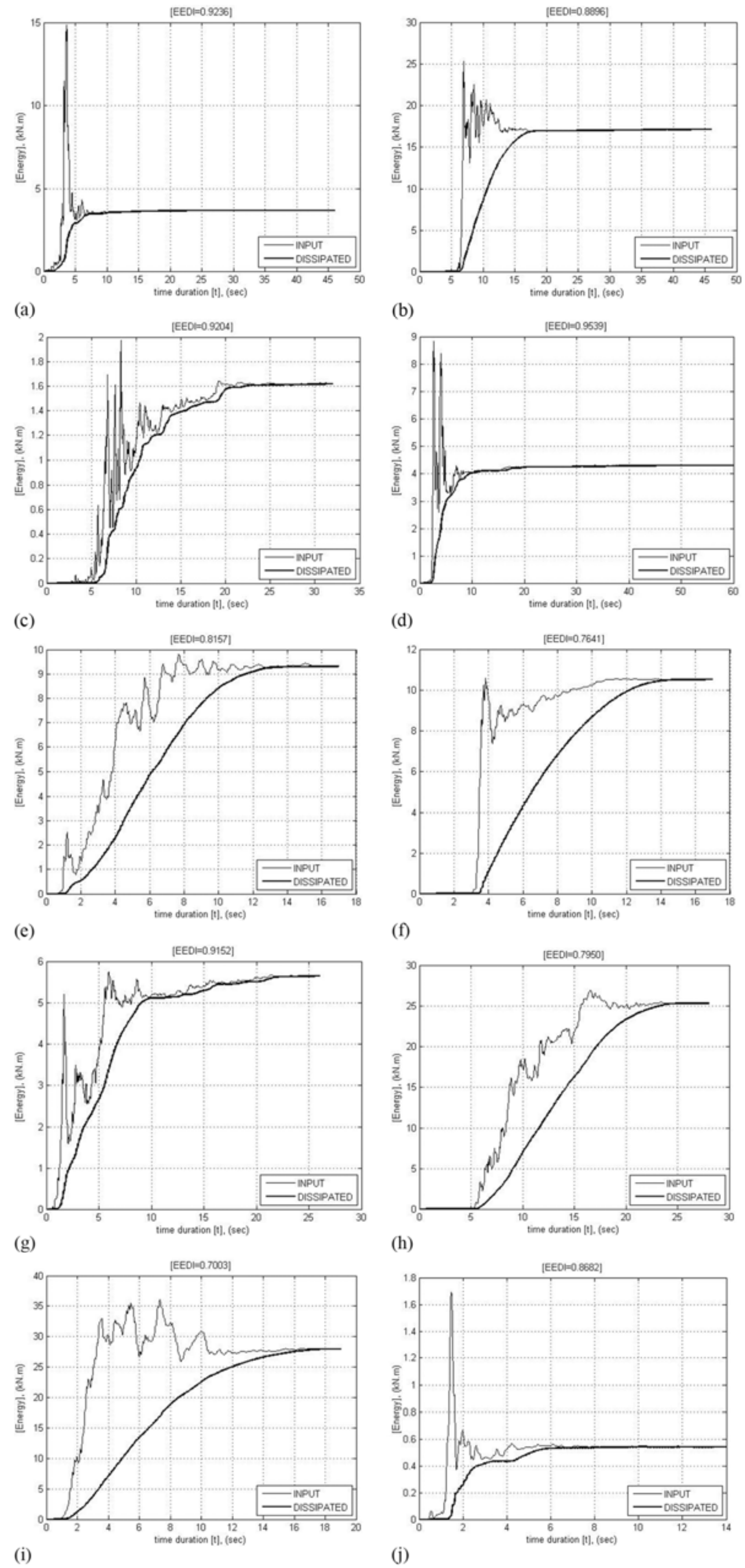

Figure 4.2. ADCS2 hysteretic damper's energy dissipation and force-deformation behaviour (damper: 616355): (a-j) seismic case 1-10

4.4. Relative Displacements

The absolute values of the relative displacements of ADCS1 under the ten seismic records of the Mediterranean are compared to the respective responses of the primary frame in Table 4.2. The results verify that in 60 % of the cases the relative displacements of the controlled system are not increased, whereas a slight increase is obtained in 40 % of the cases as follows: 0.27 % in seismic case 3, 0.77 % in case 4, 0.84 % in case 7 and 0.97 % in case 10. The respective responses decrease accounts to 2.50 % in case 1, 0.56 % in case 2, 52.34 % in case 5, 2.47 % in case 6, 70.00 % in case 8 and 23.41 % in case 9. On average ADCS1 causes 14.84 % decrease of the system's maximum relative displacements. In the case of ADCS2, favourite results are obtained in 70 % of the seismic loading cases, Table 4.2. In cases 2, 3 and 6, an increase of the response is registered, by 0.38 %, 0.36 % and 0.25 % respectively. The respective decrease accounts to 29.60 % in case 1, 31.25 % in case 4, 31.00 % in case 5, 0.90 % in case 7, 38.35 % in case 8, 0.23 % in case 9, and to 62.90 % in case 10. On average ADCS2 causes 19.32 % decrease of the system's maximum relative displacements.

Table 4.2. Primary frame's, ADCS1 and ADCS2 maximum relative displacements U_x

Seismic case	Max. relative displacement [cm]		
	Primary frame	ADCS1	ADCS2
1	1.918	1.87	1.35
2	2.494	2.48	2.59
3	0.633	0.65	0.86
4	1.542	1.66	1.06
5	2.119	1.01	1.46
6	1.661	1.62	1.62
7	1.067	1.96	0.97
8	3.066	0.91	1.89
9	2.977	2.28	2.91
10	0.629	0.69	0.45

5. CONCLUSIONS

In the present paper the concept of Adaptable Dual Control Systems is analytically investigated in two different configurations. The proposed system consists of a hysteretic damper and a bracing mechanism of closed circuit. ADCS innovative mechanism enables the elastic response of the primary structure and the dissipation of the earthquake induced energy through plastic hysteresis in the damper. The composition of the bracing-damper mechanism leads to a continuous most uniform counteraction of all structural members to resist the earthquake loading, while practically avoiding any stiffness-interaction with the primary frame. The application of the control mechanism becomes an attractive alternative, not only for the design of earthquake resistant structures, but also for the seismic retrofit of existing ones.

The systems previously investigated in their dynamic responses under three international strong earthquake motions, have been presently verified in their performance based on ten earthquake records of the Greek Mediterranean region. While the portal bracing is primarily responsible for its relative displacements to the primary system leading thus to deformations of the interconnected hysteretic damper, the additional chevron bracing has proven through its re-centring action, to enable not only further increase of the damper's deformations and the resulting energy dissipation, but also decrease of the sensitivity of the control mechanism to the earthquake loading.

REFERENCES

Aiken, I.D., Nims, D.K., Whittaker, A.S. and Kelly, J.M. (1993). Testing of passive energy dissipation systems. *Earthquake Spectra*, **9:3**, 335-369.

Constantinou, M.C. and Symans, M.D. (1993). Seismic response of structures with supplemental damping. *The Structural Design of Tall Buildings*, **2**, 77-92.
Dargush, G.F. and Soong, T.T. (1995). Behavior of metallic plate dampers in seismic passive energy dissipation systems. *Earthquake Spectra*, **11:4**, 545-568.
Di Sarno, L. and Elnashai, A.S. (2005). Innovative strategies for seismic retrofitting of steel and composite structures. *Earthquake Engineering and Structural Dynamics*, **7**, 115-135.
Filiautrault, A. and Cherry, S. (1988). Comparative performance of friction damped systems and base isolation systems for earthquake retrofit and aseismic design. *Earthquake Engineering and Structural Dynamics*, **16**, 389-416.
Kurata, M., DesRoches, R. and Leon, R.T. (2008). Cable damper bracing for partial seismic rehabilitation. *14th World Conference on Earthquake Engineering*, Beijing, China.
Mualla, I.H. and Belev, B. (2002). Performance of steel frames with a new friction damper device under earthquake excitation. *Engineering Structures*, **24**, 365-371.
Nakashima, M., Saburi, K. and Tsuji, B. (1996). Energy input and dissipation behavior of structures with hysteretic dampers. *Earthquake Engineering and Structural Dynamics*, **25:12**, 483-496.
Phocas, M.C. and Pocanschi, A. (2003). Steel frames with bracing mechanism and hysteretic dampers. *Earthquake Engineering and Structural Dynamics*, **32:5**, 811-825.
Renzi, E., Perno, S., Pantanella, S. and Ciampi, V. (2007). Design, test and analysis of a light-weight dissipative bracing system for seismic protection of structures. *Earthquake Engineering and Structural Dynamics*, **36:4**, 519-539.
Sophocleous, T. and Phocas, M.C. (2009a). Dual earthquake resistant frames. Phocas, M., Brebbia, C.A. and Komodromos, P., (eds.), *Earthquake Resistant Engineering Structures VII, Section 4: Passive Protection Devices*, WIT Press, Southampton, 165-174.
Sophocleous, T. and Phocas, M.C. (2009b). Model of analysis for earthquake resistant dual systems. *2nd International Conference on Computational Methods in Structural Dynamics and Earthquake Engineering*, Rhodes, Greece.
Sophocleous, T. and Phocas, M.C. (2010). Dual structures towards kinetic adaptability for earthquake resistance. *First International Conference on Structures & Architecture*, Guimaraes, Portugal.
Symans, M.D., Charney, F.A., Whittaker, A.S., Constantinou, C., Kircher, C.A., Johnson, M.W. and McNamara, R.J. (2008). Energy dissipation systems for seismic applications: Current practice and recent developments. *Structural Engineering*, **1:3**, 0733-9445.
Tremblay, R. and Filiatrault, A. (1996). Seismic impact loading in inelastic tension-only concentrically braced frames: myth or reality? *Earthquake Engineering and Structural Dynamics*, **25:12**, 1373-1389.
Tsai, K.C., Chen, H.W., Hong, C.P. and Su, Y.F. (1993). Design of steel triangular plate energy absorbers for seismic-resistance construction. *Earthquake spectra*, **9**, 505-528.
Wu, B., Zhang, J., Williams, M.S. and Ou, J. (2005). Hysteretic behavior of improved Pall-typed frictional dampers. *Engineering Structures*, **27**, 1258-1267.
Xia, C. and Hanson, R.D. (1992). Influence of ADAS element parameters on building seismic response. *Structural Engineering*, **118:7**, 1903-1918.