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Parametric Non-Linear Analysis of Reinforced Concrete Frames with Supplemental Damping Systems

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***Abstract*—**This paper focuses on parametric analysis of reinforced concrete structures equipped with supplemental damping braces. Practitioners still luck sufficient data for current design of damper added structures and often reduce the real model to a pure damper braced structure even if this assumption is neither realistic nor conservative. In the present study, the damping brace is modelled as made by a linear supporting brace connected in series with the viscous/hysteretic damper. Deformation capacity of existing structures is usually not adequate to undergo the design earthquake. In spite of this, additional dampers could be introduced strongly limiting structural damage to acceptable values, or in some cases, reducing frame response to elastic behavior. This work is aimed at providing useful considerations for retrofit of existing buildings by means of supplemental damping braces. The study explicitly takes into consideration variability of (a) relative frame to supporting brace stiffness, (b) dampers' coefficient (viscous coefficient or yielding force) and (c) non-linear frame behavior. Non-linear time history analysis has been run to account for both dampers' behavior and non-linear plastic hinges modelled by Pivot hysteretic type. Parametric analysis based on previous studies on SDOF or MDOF linear frames provide reference values for nearly optimal damping systems design. With respect to bare frame configuration, seismic response of the damper-added frame is strongly improved, limiting deformations to acceptable values far below ultimate capacity. Results of the analysis also demonstrated the beneficial effect of stiffer supporting braces, thus highlighting inadequacy of simplified pure damper models. At the same time, the effect of variable damping coefficient and yielding force has to be treated as an optimization problem.

***Keywords*—**Brace stiffness, dissipative braces, non-linear analysis, plastic hinges, reinforced concrete.

I. INTRODUCTION

INNOVATIVE strategies for controlling excessive vibrations induced by earthquake loads in new building structures (new design) as well as in existing ones (retrofit design), include use of supplemental energy dissipation systems.

Viscous and hysteretic dampers are generally attached to steel supporting braces. Usually, effective stiffness of supporting braces is neglected thus introducing some approximation.

In this paper, the influence of damping coefficient and supporting brace stiffness on the dynamic response of multi-story buildings is properly considered.

Although the adoption of supplemental damping devices for seismic protection of buildings is no more a novelty for earthquake engineers, a clear design methodology for the selection of the mechanical parameters to be assigned to the energy dissipaters is not yet clearly available. However, some authors have tackled the problem and proposed some design procedures, each referred to a specific energy dissipation device [1]-[5].

In [6], a design procedure is proposed to determine the optimal design parameter of dissipative braces installed in a simple one story–one bay frame, whose behavior has to remain elastic. External supplemental devices are assumed to be viscous dampers (linear viscous behavior) or friction dampers (rigid plastic behavior). The suggested methodology provides useful charts to understand the best design choice of suitably defined dimensionless damping parameters (i.e., dimensionless viscous damping and dimensionless yielding displacement), and points out the fundamental influence of a properly dimensioned frame to brace stiffness ratio on the optimal design of the dissipation devices, while current state of art usually assumes the supporting brace as infinitely rigid and models also the damper as directly connecting two stories. Theoretical optimal damping parameters were provided as the ones corresponding to a minimum of the resonance peak frame displacement and base shear, in the overall range of frequencies. Theoretical results were validated by means of numerical integration of the framed structure under real ground motions, thus giving just the required effectiveness to the design procedure. A similar approach had also been adopted by the authors for dimensioning the optimal seismic protection systems for isolated bridges [7].

Most of the seismic design procedures for buildings are essentially related to concepts of performance-based and damage-controlled design. In this perspective, provision of additional dampers for damage protection of existing buildings may represent a well suited solution. For existing buildings in high seismic prone areas, non linear behavior may be also triggered in case of well designed additional dampers. In this case, the aim of the provided control system is mainly reduction of damage.

Nowadays, the design of non linear frames in conjunction with dissipative systems still needs to be properly addressed. Dissipative braces represent an effective solution for seismic retrofit of existing buildings but usually, especially for higher hazard levels, to keep the framing system into linear range may be hard to accommodate. For this reason, the designer may need to properly model non linear frame behavior in

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addition to the damping systems, the latter having to reduce as much as possible framing damage. One of the reasons why this solution is still lacking in common practice is due to toughness of modeling with respect to both available commercial software and common practitioners skills.

Some authors tried to provide simplified analysis methods for analysis of non linear damping braced frames [8], [9], trying to reduce the problem by defining a suitable strength reduction factor.

Starting from the outcome of [6], in this paper, the authors perform a set of parametric nonlinear direct integration analysis on a non-linear reinforced concrete frame equipped with additional supplemental dampers.

Aim of the authors is to determinate the effectiveness of additional damping systems in reducing structural response, and above all, damage. In this perspective, different configurations with variable supporting brace stiffness and damping coefficient have been analyzed by means of time history analysis in SAP2000.

II. Case Study

The case under study represents an existing 4-storey building designed for gravity loads only in Italy before the '80s. For the aim of the work, a plane longitudinal 5-bay frame is extracted from the 3D structure (Fig. 1) and assumed to be retrofitted by means of supplemental dampers.

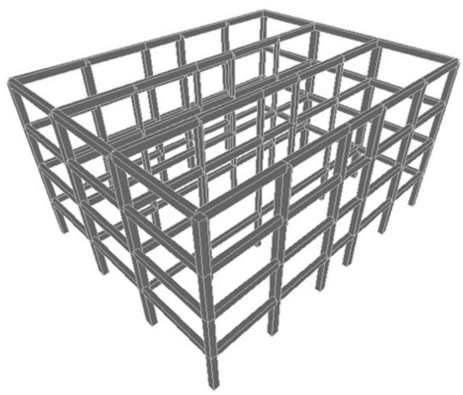

Fig. 1 3D building with 2D longitudinal frame under study

It is a symmetric 4-storey 5-bay frame, characterized by an inter-storey drift of 3.2 m and a span length of 5 m for lateral bays and 2.9 m for the central one. Considering a tributary area of 5 m, on each floor a concentrated mass of 60 tons is applied. Columns cross sections are 40x40.35x35 and 30x30 cm^2 at ground floor, while at the upper floors cross sections are 30x30 cm^2. All the beams have cross section 30x50 cm^2.

A preliminary modal analysis conducted on the structure showed a first translational vibration mode with a period T=0.83 s and a participant mass of 82%.

By applying a triangular force distribution to the frame, the lateral storey stiffness distribution in Table I was obtained.

A. Brace-Damper Compliance

Brace distribution along the frame is assumed as the one shown in Fig. 2, where four damper braces are positioned on each floor in a diagonal configuration. This assumption clearly affects the overall response, and the brace distribution is usually defined according to architectural reasons.

TABLE I
LATERAL STIFFNESS FRAME DISTRIBUTION

$k_{f,1}$ [kN/m]	54526
$k_{f,2}$ [kN/m]	26754
$k_{f,3}$ [kN/m]	26119
$k_{f,4}$ [kN/m]	25773

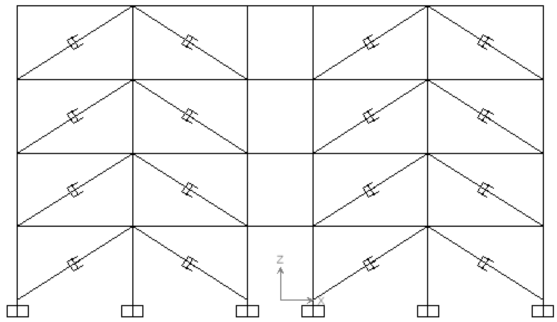

Fig. 2 2D frame with damper braces

A normalized stiffness parameter $\kappa = k_{f1}/k_{b1}$ is introduced to take into account the brace to frame stiffness. Two different brace configurations are examined (Table II): an "equal" configuration (E) with $k_{bi} = k_{b1} = k_{f1} \cdot \kappa$, and a "proportional" configuration (P), with $k_{bi} = k_{fi} \cdot \kappa$. In the following, three different values of κ (0.1, 0.5, 1) have been assumed in order to investigate the effect of the relative brace to frame stiffness, whit different level of damping.

TABLE II
BRACE STIFFNESS CONFIGURATIONS

	κ = 0,1		κ = 0,5		κ = 1,0	
	E	P	E	P	E	P
$k_{b,1}$ [kN/m]	545260	545260	109052	109052	54526	54526
$k_{b,2}$ [kN/m]	545260	267540	109052	53508	54526	26754
$k_{b,3}$ [kN/m]	545260	261190	109052	52238	54526	26119
$k_{b,4}$ [kN/m]	545260	257730	109052	51546	54526	25773

The brace-damper device is made of a linear elastic element connected in series with a linear viscous dashpot (viscous case) or a rigid perfectly plastic element (friction case).

Bare frame will be denoted by case A, assuming 5% damping ratio. In addition to this, a case with linear elastic brace and 5% damping will be also introduced as case Z for each value of κ.

Additionally, for each value of κ, several cases have been considered with different values of the damping parameter (cases B, C, D for viscous case; cases B, C, D, E for friction case), including both equal and proportional brace stiffness distribution.

B. Non Linear Plastic Hinge

Frame behaviour was assumed be non linear to properly investigate the combined influence of additional dampers. Mean values of concrete and steel strength are f_{cm}=17.5 MPa and f_{ym}=350 MPa, respectively, assuming low ratio internal reinforcement.

The structure has been modelled through SAP2000 software, adopting a plastic hinge model for definition of flexural non linearities at ends cross sections.

A bilinear elastic-perfectly plastic moment rotation relation $M-\theta$ was assumed as in Fig. 3.

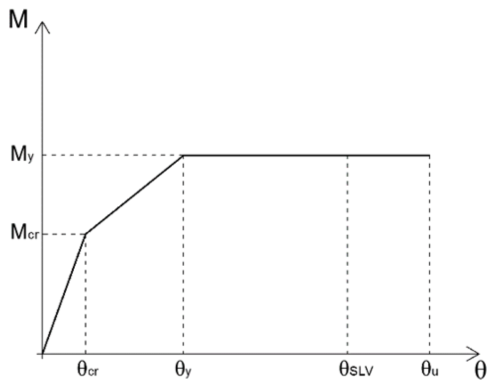

Fig. 3 $M-\theta$ relation

The $M-\theta$ relation is completely defined through four characteristic points including i) cracking for θ_{cr}, ii) yielding for θ_y, iii) life safety limit state for $\frac{3}{4}\ \theta_u$ and iv) collapse for θ_u. Values of θ_y and θ_u are obtained according to [10], [11].

A Pivot hysteresis model is introduced to adequately capture the nonlinear behavior of reinforced concrete members. Reference [12] demonstrated that results based on the proposed hysteresis model are in agreement with experimental ones. This method is based on the observation that unloading and reverse loading tend to be directed toward specific points, called "Pivots points" [13] on the force-deformation (or M-θ) plan. The hysteresis model is defined by means of three additional scalar parameters: α and β are defined as a function of axial load and longitudinal steel ratio, whereas η is a function of stiffness degradation.

The exact definition of the aforementioned parameters would require a direct experimental investigation. For parameters calibration, data from a sample of laboratory tests on reinforced concrete columns with rectangular cross section were considered [14]. By means of visual inspection of experimental results, parameters $\alpha = 10$, $\beta = 0{,}5$ and $\eta = 10$ have been set (Fig. 4).

C. Design Seismic Action

According to the Italian building code [10], the design spectra (Fig. 5) with 5% of critical damping have been defined for the collapse prevention limit state (SLC) of a conventional building (functional class II) located in Sant'Angelo dei Lombardi (AV), Italy (15.18° longitude, 40.93° latitude) on soil type B (360≤Vs,30≤800 m/s) with a nominal life of 50 years, corresponding to a return period of 975 years, and providing a Peak Ground Acceleration equal to 0.46 g. A set of seven unscaled accelerograms matching the reference spectrum (Fig. 5, Table III) was found in the European ground motion database using Rexel v3.4 beta [15]. The average spectrum has 10% lower and 30% upper tolerance in the period range 0.15-2 s.

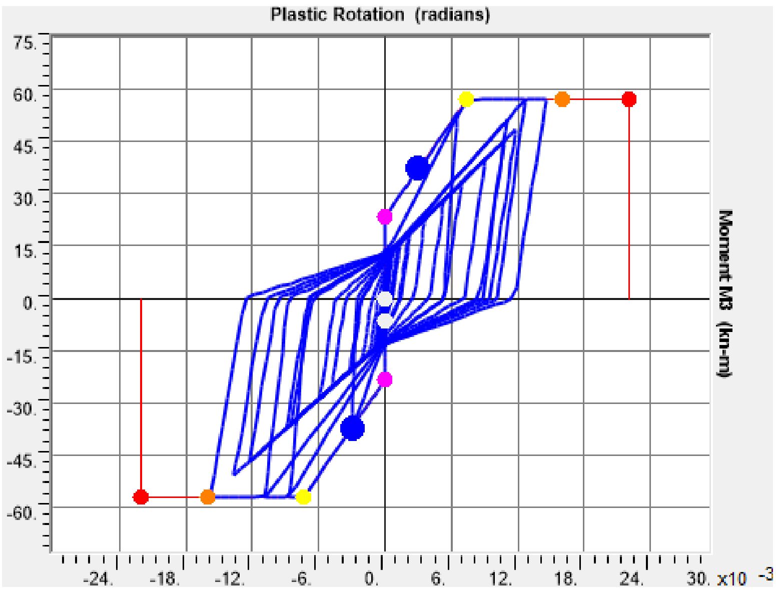

Fig. 4 Hinge Results for Pivot hysteretic model

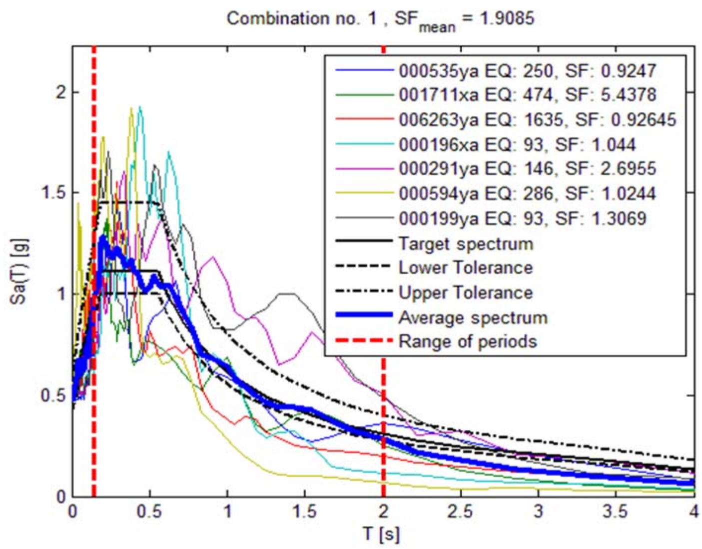

Fig. 5 Design spectrum for SLC

TABLE III
GROUND MOTION SELECTION

Waveform ID	Station ID	Earthquake Name	Mw	PGA [m/s^2]
535	ST205	Erzincan	6.6	5.0
1711	ST1255	Ano Liosia	6	0.9
6263	ST2484	South Iceland	6.5	5.0
196	ST62	Montenegro	6.9	4.5
291	ST276	Campano Lucano	6.9	1.7
594	ST60	Umbria Marche	6	4.5
199	ST67	Montenegro	6.9	3.6

III. VISCOUS DAMPERS

For each of the assumed values of κ, a pattern of viscous dampers'coefficients (Tables IV-VI) has been selected according to [16]. In [16], the optimal damping coefficient is obtained in order to minimize the top storey drift for a MDOF linear frame. In this case, the same values are adopted even assuming a non linear frame behaviour. It can be noted that damping coefficient usually reduces with increasing κ, i.e. for larger flexibility of supporting braces.

TABLE IV
VISCOUS DAMPER COEFFICIENTS FOR $\kappa = 0.1$

Case	κ [-]	c_h [kN s/m]	c_{diag} [kN s/m]	c_{diag} / 4 [kN s/m]
A		0	0	0
B_E		2984	3552	888
B_P		2453	2920	730
C_E		5968	7105	1776
C_P	**0.10**	4906	5840	1460
D_E		11936	14210	3552
D_P		9812	11681	2920
Z_E		∞	∞	∞
Z_P		∞	∞	∞

A "damper exponential" non linear link is introduced in the FEM model, considering a viscous damper in series with an elastic brace. Dampers are conveniently considered equal on each floor.

TABLE V
VISCOUS DAMPER COEFFICIENTS FOR $\kappa = 0.50$

Case	κ [-]	c_h [kN s/m]	c_{diag} [kN s/m]	c_{diag} / 4 [kN s/m]
A		0	0	0
B_E		2450	2917	729
B_P		1846	2198	549
C_E		4900	5833	1458
C_P	**0.50**	3692	4395	1099
D_E		9800	11667	2917
D_P		7384	8790	2198
Z_E		∞	∞	∞
Z_P		∞	∞	∞

TABLE VI
VISCOUS DAMPER COEFFICIENTS FOR $\kappa = 1.00$

Case	κ [-]	c_h [kN s/m]	c_{diag} [kN s/m]	c_{diag} / 4 [kN s/m]
A		0	0	0
B_E		1837	2187	547
B_P		1270.5	1513	378
C_E		3674	4374	1093
C_P	**1.00**	2541	3025	756
D_E		7348	8748	2187
D_P		5082	6050	1513
Z_E		∞	∞	∞
Z_P		∞	∞	∞

A. Analysis Results

Non linear time history analysis has been run for each brace-damper configuration under the set of ground motions, in order to estimate the average response as representative of a design value.

Displacement at top (Fig. 6) and base shear (Fig. 7) are considered for each configuration and are compared with case A. Axial force (Fig. 8) in a base column is also monitored to detect effects of additional braces. At the same time, plastic rotations at some hinge locations (Figs. 9-11) have been compared with capacity values.

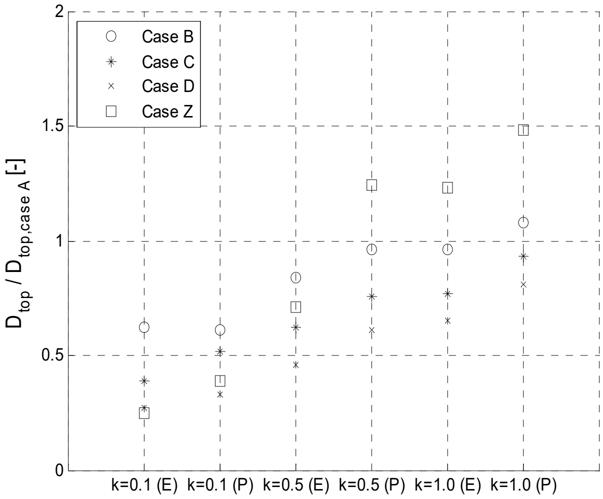

Fig. 6 Viscous case: displacement results

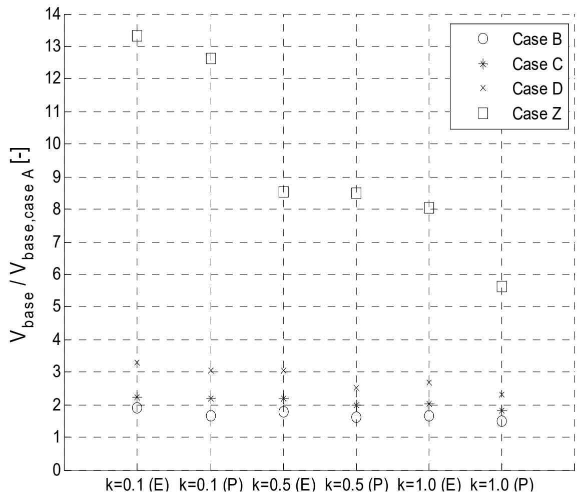

Fig. 7 Viscous case: base shear results

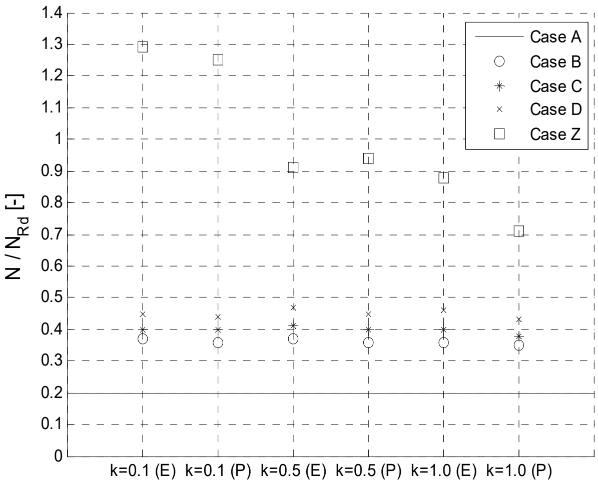

Fig. 8 Viscous case: axial force in a base column

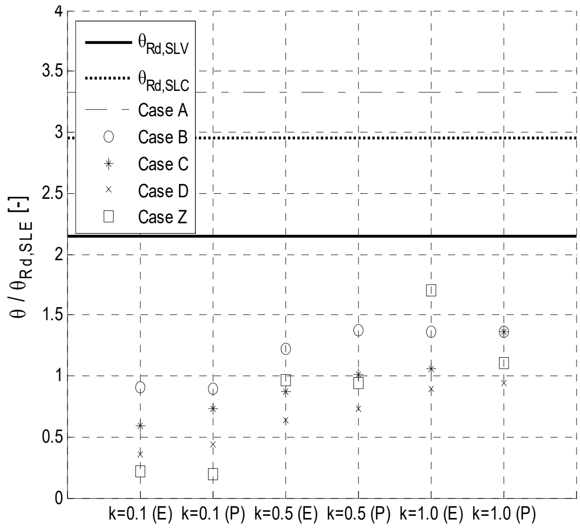

Fig. 9 Viscous case: plastic hinge rotations for column P 1_7

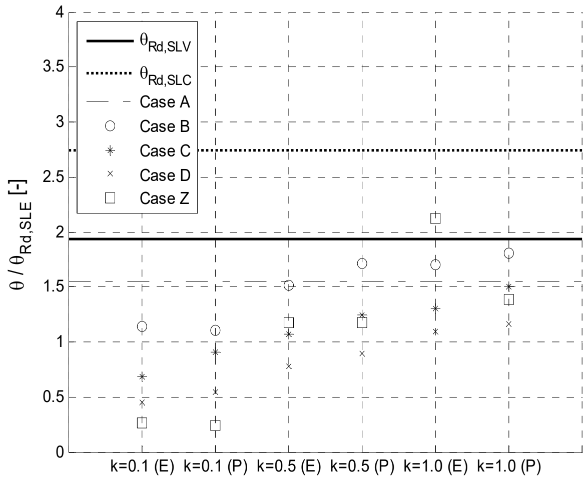

Fig. 10 Viscous case: plastic hinge rotations for column P 1_8

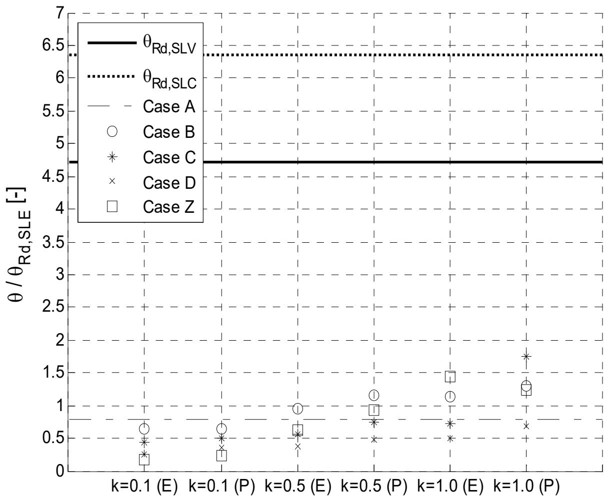

Fig. 11 Viscous case: plastic hinge rotations for beam T 1_7-8

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In all cases where braces have been introduced, base shear increases with respect to case A. An important outcome is that, in cases B and , base shear increase is approximately twice the bare frame, while case Z would be much higher. In spite of this, a great advantage is obtained in terms of drift reduction and, consequently, plastic rotations. A strong sensitivity of structural response to the value of k can be noted in terms of drift reduction and plastic rotations: an almost linear increasing trend can be observed with increasing flexibility of the supporting brace. For a given value of k, as far as the damper constant value is concerned, the higher the viscous coefficient, the higher the base shear and the lower the top displacement. Very high value of damping constant (case D) may be avoided to limit the base shear that may peak around three times case A.

Case Z is usually among the most beneficial in terms of deformability, but it provides very large increment of base shear and axial force, due to limited value of damping despite provided additional stiffness. A part from this, only in the same cases, case Z corresponds to minimum top displacement. A general outcome is that dampers are beneficial for low values of k: for $k = 1$ drift reduction is minimal, and plastic rotations tend to case A. In terms of damage, it must be also said that cases with $k < 0.5$, and higher damping coefficients are able to provide rotations lower than yielding point, thus meaning that no damage occurs in the main frame.

In terms of brace stiffness, it can be said that for $k = 0.1$, displacements have the maximum reduction, while base shear is lightly affected by different values of k. Case with $k = 1$ provided poorer effect in terms of drift reduction mainly contributing to increase of base shear.

IV. Friction Dampers

The yielding force in the devices has been obtained by reducing the real frame to an equivalent SDOF and applying the procedure suggested by [6].

The base shear of the equivalent SDOF at the yielding point of the device is provided by (1).

$$F_y = \alpha \cdot \delta_{opt} \cdot m \cdot a_g \tag{1}$$

where δ_{opt} is provided by the procedure, m is the first mode participating mass, a_g is the PGA and α is a calibration parameter.

The equivalent yielding force of the device is expressed by (2).

$$F_c = \frac{F_y}{1+\kappa} \tag{2}$$

The yielding force in each device $F_{c,i}$ (Table VII) is assumed as (3).

$$F_{c,i} = F_c \cdot \frac{F_{y,i}}{F_y^*} \tag{3}$$

where $F_{y,i}$ is the story shear at 0,5% inter storey drift (Table VIII) and F_y^* the corresponding base shear in the bare frame.

A non linear "Plastic (Wen)" link has been introduced to model the friction device mounted in series with the elastic brace.

TABLE VII
Friction Damper Yielding Force Distribution

	B_E	B_P	C_E	C_P	D_E	D_P	E_E	E_P
α=	0.125		0.25		0.5		1	
$F_{c,1}$ [kN]	281	281	562	562	1124	1124	2249	2249
$F_{c,2}$ [kN]	253	253	506	506	1012	1012	2024	2024
$F_{c,3}$ [kN]	197	197	394	394	787	787	1574	1574
$F_{c,4}$ [kN]	113	113	225	225	449	449	899	899

TABLE VIII
Frame Yielding Force Distribution

$F_{y,1}$ [kN]	445.9
$F_{y,2}$ [kN]	401.3
$F_{y,3}$ [kN]	312.1
$F_{y,4}$ [kN]	178.4

A. Analysis Results

Results of time history analysis are reported in Figs. 12-17.

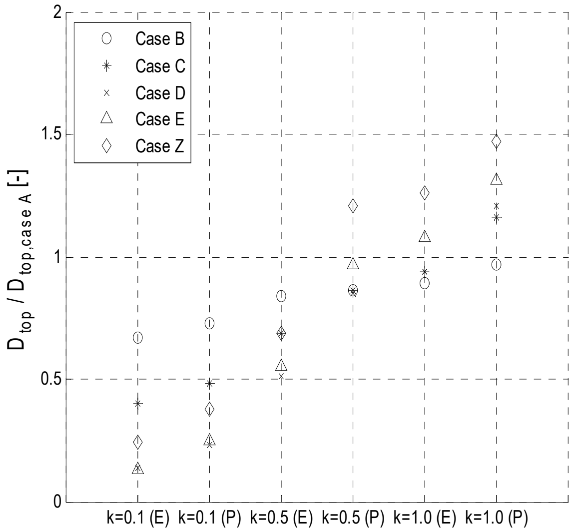

Fig. 12 Friction case: displacement results

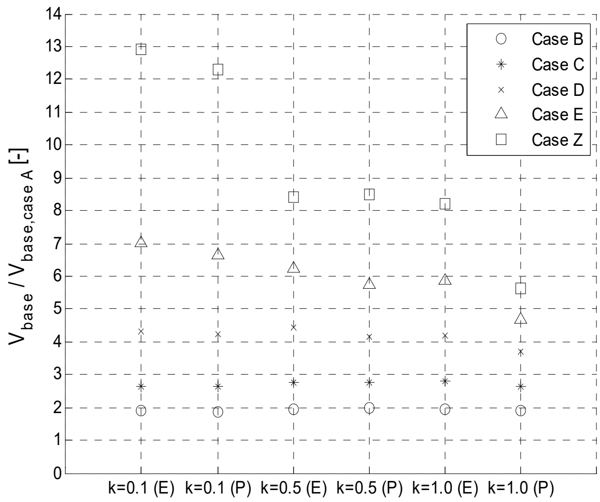

Fig. 13 Friction case: base shear results

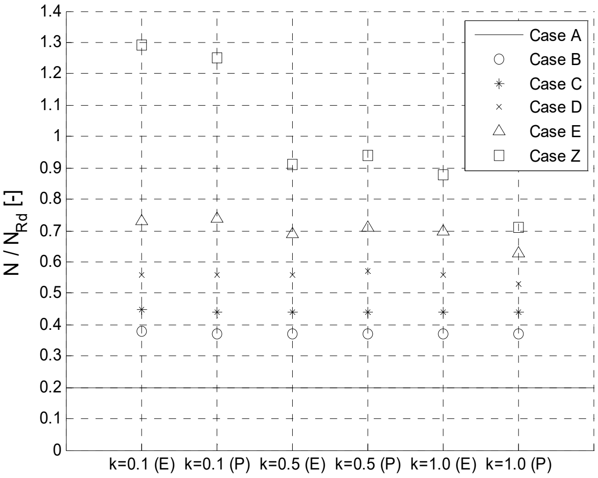

Fig. 14 Friction case: axial force in a base column

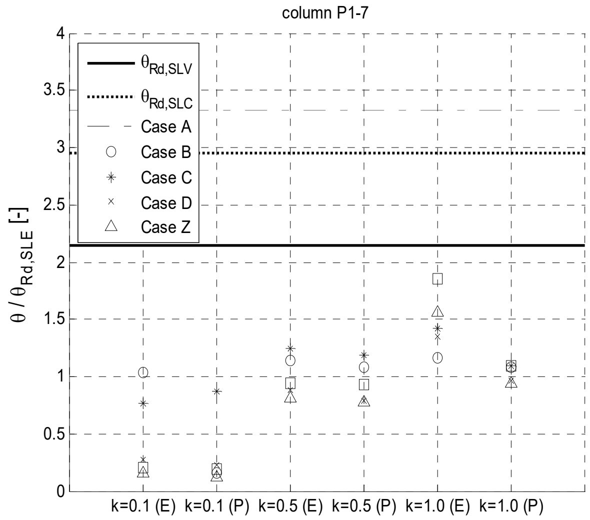

Fig. 15 Friction case: plastic hinge rotations for column P 1_7

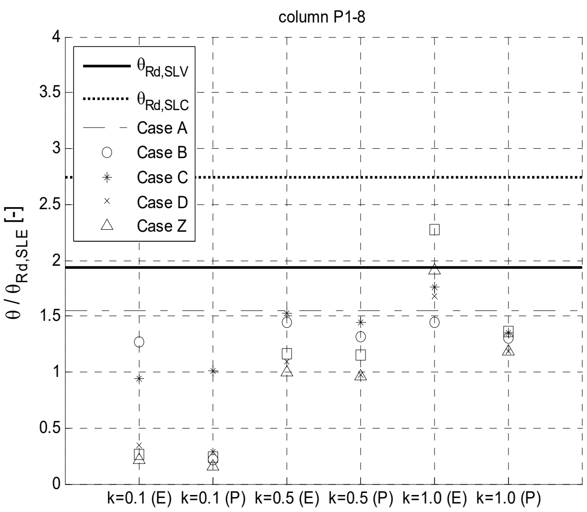

Fig. 16 Friction case: plastic hinge rotations for column P 1_8

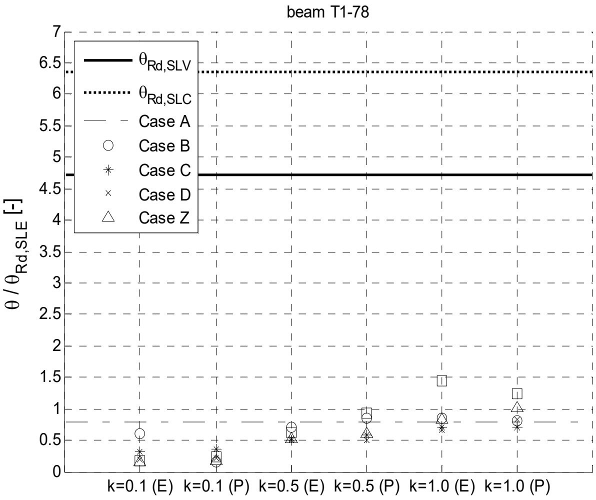

Fig. 17 Friction case: plastic hinge rotations for beam T 1_7-8

With respect to viscous case, the following considerations can be drawn. Friction devices usually provide higher values of base shear and axial force in the columns. Also in this case, a stronger sensitivity of structural response to the value of k can be noted in terms of drift reduction and plastic rotations and that cases with $k < 0.5$, and well tuned yielding force can strongly limit or even prevent plastic deformations.

As far as the friction parameter is concerned, the higher the yielding force, the higher the base shear, and the lower the top displacement.

Higher value of yielding force (case E) tends to provide a global response similar to case Z whit higher base shear, due to limited plastic excursion of dampers.

Minimum displacements do not correspond to case Z, thus demonstrating the stronger effectiveness of damping with respect to stiffening effect.

V. Conclusion

This paper proposed a parametric investigation of non linear reinforced concrete structures equipped with supplemental damping braces.

In the current study, the effect of flexible viscous or hysteretic dampers was explicitly combined with non linear frame behaviour. Values of damper devices were selected from suggested optimal design procedures.

Non linear time history analysis was run to properly consider the combined effect of deformable added dampers and non linear plastic hinges. It was demonstrated that, by accurate selection of damping braces' parameters, seismic response of the frame may be strongly improved, limiting plastic deformations to acceptable values far below ultimate capacity.

A general outcome is that dampers are more beneficial for low values of k, i.e. stiffer supporting braces. For $k = 1$ drift and plastic hinge reduction was minimal whereas base shear increased.

In terms of stiffness effect, it can be said that for $k = 0.1$, displacements have the maximum reduction while base shear is lightly affected by different values of k. Minimum displacements do not correspond to case Z, thus demonstrating stronger effectiveness of additional damping with respect to stiffening effect.

Cases with $k = 1$ provided poorer effects in terms of drift reduction mainly contributing to increase of base shear.

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