Plastic hinges modification factors for damaged RC columns

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15 WCEE
LISBOA 2012

SUMMARY:
The behavior of damaged buildings can be simulated with a suitable modification of plastic hinges for damaged elements, by means of stiffness and strength degradation and considering possible residual drifts. There exist some proposals in literature that give, depending on damage severity, component modification factors. However, those methods are referred to reinforced concrete (RC) members typically found in North American or Japanese buildings (i.e. walls, piers), which are not representative of under-designed columns typically found in Mediterranean European regions. This paper presents the experimental-based calibration of modification factors for plastic hinges of damaged columns representative of existing buildings with design characteristics non-conforming to present-day seismic provisions.

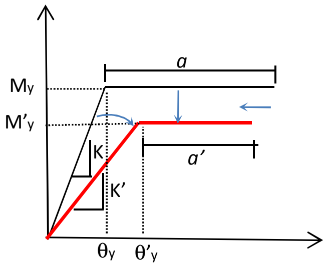
Keywords: modification factors, stiffness, strength, residual drift, plastic hinge, RC column

1. INTRODUCTION

The behavior of earthquake damaged buildings may be simulated with a suitable modification of plastic hinges for damaged elements. Such a modification is based on stiffness, strength and displacement reduction factors, λ_k, λ_Q and RD, respectively (see Fig. 1), accounting for the achieved damage states on the structural elements. Based on the type of elements and observed behavior (e.g. pure flexural, flexure-shear, sliding shear etc.), and considering the relative gravity of damage, suitable factors to be applied for modification of plastic hinges in the damaged models may be suggested. In (FEMA 1998a,b) values of λ and RD are proposed for various element typologies and behavioral modes; these values are based on experimental calibration and/or on theoretical derivation.
Japanese guidelines for the assessment of buildings capacity in the post-earthquake (JBDPA, 1991; Nakano et al., 2004) suggest a method which takes into account the variation of a seismic capacity index depending on observed damage severity. In these guidelines, the building occupancy assessment depends on the variation of index I_s in the pre-earthquake and post-earthquake stage; I_s is proportional to the product of a strength index C (i.e. base shear) and a ductility index F, representative of the building deformation capacity. In particular, in order to assess post-earthquake condition, a residual capacity percentage index, R, is defined as follows:

$$R = \frac{I_{S,D}}{I_S} \cdot 100\,(\%) \tag{1.1}$$

where $I_{s,D}$ is the seismic index on the damaged structure. $I_{s,D}$ can be computed on the basis of a capacity reduction factor, η, depending on the structural elements hysteretic dissipation capacity in the pre and post-earthquake stage.



	stiffness	strength	Plastic rotation
intact	K	M_y	a
damaged	$K' = \lambda_K K$	$M_y' = \lambda_Q M_y$	$a' = a - a_d$

Figure 1. Modeling criteria for the damaged plastic hinges (after FEMA 1998a)

Japanese Guidelines suggest different η values, calibrated on experimental tests (Maeda et al. 2004), depending on damage severity level for different element typologies (brittle or ductile columns, walls etc). They also emphasize that a wider range of experimental tests are necessary to better calibrate the member residual capacity. The Japanese approach, based on the factor η, is conceptually similar to that reported in (FEMA, 1998a), based on suitable modification of plastic hinges for damaged elements, λ and *RD*. The former values of η (or λ, *RD*) are mainly representative of RC members such as walls or strong piers that can be typically found in Japanese (or North American) buildings. The few indications that may be found for RC columns cannot be indiscriminately used for RC members typical of European Mediterranean regions, because their mechanical properties, the type of reinforcement (smooth or deformed bars) and the relative percentage as well as type of detailing, may differ significantly from those of North America or Japan. Therefore, there is a need for proper calibration of modification factors for plastic hinges of damaged columns representative of existing elements with design characteristics non-conforming to present-day seismic provisions. In order to propose theoretical expressions to compute these factors, the present paper focuses on their calibration based on the available literature experimental results.

A preliminary application, adopting modification factors calibrated on a part of the full database adopted in this paper, may be found in (Polese et al, 2012), where the variation of an existing RC building residual capacity depending on the damage state caused by a potential main-shock is investigated.

2. EXPERIMENTAL DATABASE: SELECTION CRITERIA

The available database used in this study is the Pacific Earthquake Engineering Research Center's Structural Performance Database (PEER, 2005), developed at the University of Washington by Berry, Parrish, and Eberhard (Berry et al., 2004). The database includes the results of 416 tests on square (209 tests), rectangular (44 tests) or circular (163 tests) RC columns under axial load and uniaxial bending provided by monotonic or cyclic horizontal actions. The available database was enriched by means of experimental tests performed at University of Naples Federico II; in particular, 14 tests under monotonic or cyclic actions on square or rectangular RC columns designed according to provisions, construction practice and material properties enforced between '40s and '70s years in Italy have been added to existing results (Di Ludovico et al., 2009; Verderame et al., 2008a, b).

Tests results on square or rectangular columns reinforced with deformed rebars, under cyclic actions have been initially selected for the present study; therefore a new database made of 253 tests was obtained.

In order to calibrate plastic hinges modification factors for old-type columns designed non-conforming to present-day seismic codes and practices, the paper deals with experimental data on full-scaled RC columns designed according to obsolete codes with sub-standard structural detailing. Thus, the experimental tests on columns characterized by poor confinement (square or rectangular hoops, 101 tests on 253) "nonconforming" to present day seismic codes (ASCE/SEI 41-06, 2007, hoops spacing, *s*, higher than *d/3*, with *d* effective cross section depth, 73 tests on 253) have been firstly selected. Further, to be representative of existing buildings, only columns tested under a constant normalized axial load, $\nu<0.5$ ($\nu = N/(A_c f_{cm})$, where *N* is the axial load, *Ac* is the concrete gross area, and *fcm* is the mean cylindrical concrete strength) have been considered in the calibration of modification factors

(221 tests on 253). Finally, the experimental tests governed by shear failure mode were neglected. To select flexural or combined flexure-shear collapse mode, ASCE/SEI 41-06, 2007 criteria were adopted (227 tests on 253). For columns with transverse reinforcement having 135° hooks, pure flexural failure (*condition i*) is reached if $V_p/(V_n/k) \leq 0.6$, where V_p is the demand on the column, V_n is the nominal shear strength, and k is a modifier based on ductility demand; flexure-shear failure (*condition ii*) if $0.6 \leq V_p/(V_n/k) \leq 1.0$; shear failure if $V_p/(V_n/k) \geq 1.0$ *(condition iii)*. Further, *condition i* is limited to columns with a transverse reinforcement ratio $Av/b_w s$ (with A_v cross sectional area of transverse reinforcement, b_w cross-sectional width and s transverse reinforcement spacing) greater than or equal to 0.002 and a spacing to depth ratio less than 0.5. In the case of columns with 90-degree hooks transverse reinforcement, *condition i* is adjusted to *condition ii.*
According to these criteria a total number of 18 tests have been used in the next sections for the proper calibration of modification factors of RC square or rectangular columns reinforced with deformed rebars representative of existing buildings. The main parameters collected are summarized for each test in Table 1 (at the end of paper).

3. EXPERIMENTAL PARAMETERS

The modification factors can be gathered from individual cyclic tests by examining the change in force-displacement response from cycle to cycle. In particular, initial cycles can be considered representative of the behavior of intact elements, whereas subsequent cycles for the damaged component (FEMA, 1998a). Therefore, based on the selected experimental available 18 tests, the latter procedure has been used to calibrate stiffness, strength and residual drift modification factors. The main parameters derived from experimental tests and involved in the analysis are presented in the following and summarized in Table 1. To determine the column flexural capacity, the effective horizontal force, $F_{eff.}$, applied on the column has been determined for each test as:

$$F_{eff} = \frac{M_{base}}{L_S} = \frac{F \cdot L_S + N \cdot \Delta}{L_S} \tag{3.1}$$

where M_{base} is the column base bending moment, F is the lateral applied load, N is the applied axial load, and Δ is the lateral column displacement. The column chord rotation has been assumed equal to drift: $\theta = \Delta / L_S$.
The envelope curve has been obtained for each experimental cyclic test according to the approach proposed in (Elwood et al., 2007) (i.e. by connecting the first cycle peak point for each loading step, see Figure 2 (a). On the envelope curve the following parameters have been selected:

- $F_{max.}$: maximum force attained in the test;
- θ_{Fmax}: rotation corresponding to the maximum experimental force with respect to both positive and negative loading actions, Figure 2 (a);
- θ_y: yield rotation, defined according to experimental practice proposed by (Elwood et al., 2009), see Figure 2 (b); the procedure requires the use of envelope curve. First it is necessary to determine the line passing through the intersection point between envelope curve and the horizontal line through F_y (force at which the tension reinforcement yields or the maximum concrete strain reaches a value of 0,002) and the origin; then the intersection between this line and the horizontal one through $F_{0,004}$ (force at which the strain of 0,004 is reached in the concrete) gives the yield, Figure 2 (b);
- θ_u : ultimate rotation, defined as the rotation at failure condition, set at 20% drop of the maximum lateral load, $0.8F_{max}$ (Fardis and Biskinis, 2003), see Figure 2 (a).

In order to assess the cyclic column degradation, both peak drift, θ_i and residual drift, RD_i, were evaluated for each cycle with respect to positive and negative load actions (see Figure 2 (c)); *RD* values for each cycle were determined as the drift for which F_{eff} is equal to zero, see Figure 2 (c).
The column experimental stiffness at each cycle, $k_{p\text{-}p}$, was defined as the slope of the straight line joining positive and negative peak displacement (Fig. 2(d)); this experimental peak to peak stiffness has been computed at each cycle according to the following expression:

$$k_{p-p} = \frac{F_i^+ + \left|F_i^-\right|}{\theta_i^+ + \left|\theta_i^-\right|} \tag{3.2}$$

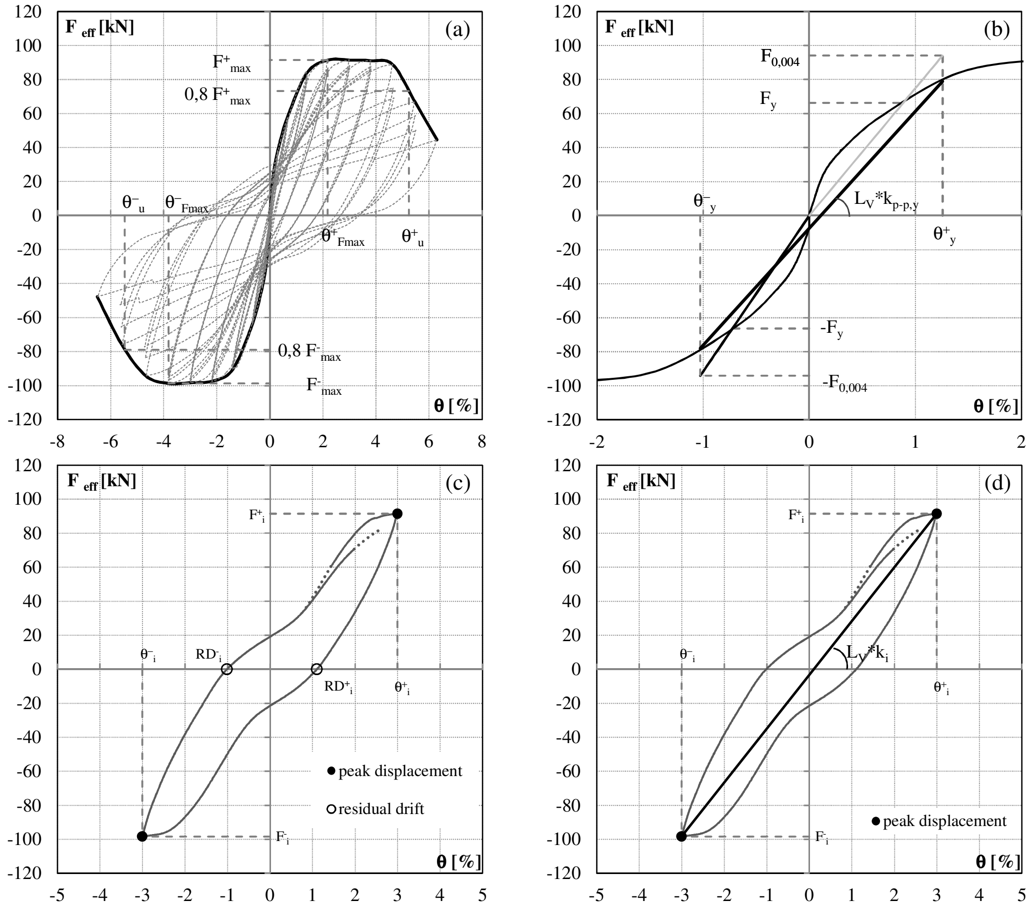


Figure 2. Experimental force-drift envelope curve (a); experimental yield rotation, θ_y, and yielding stiffness, $k_{p\text{-}p,y}$ (b); peak drift, θ_i and residual drift, RD_i, at i[th]-cycle (c); peak to peak stiffness, $k_{p\text{-}p}$ at i[th]-cycle (d).

4. STIFFNESS MODIFICATION FACTOR

In order to compute the stiffness degradation on the damaged members, a proper stiffness modification factor has been introduced, λ'_k. It has been defined as the ratio between the peak to peak experimental stiffness, $k_{p\text{-}p}$, and the experimental yield stiffness, $k_{p\text{-}p,y}$, computed as the slope (in the F_{eff}-Δ reference system) of the line joining intersections between yield rotation, θ_y, and the envelope curve (Fig. 2 (b)).

$$\lambda'_k = \frac{k_{p-p}}{k_{(p-p),y}} \tag{4.1}$$

The experimental stiffness has been normalized with respect to $k_{p\text{-}p,y}$ to compare the experimental data resulting from columns with different geometrical and mechanical properties. The experimental values of λ_k as a function of the ratio θ/θ_y are reported in Figure 3 (a). The parameter θ/θ_y has been adopted in order to correlate the stiffness degradation to the ductility level attained by the column after the damage. Note that θ has been computed at each drift level as the average peak positive and negative drift, $\theta = (\theta_i^+ + \left|\theta_i^-\right|)/2$, while θ_y has been conservatively assumed as the minimum yield rotation experienced in positive and negative load actions. Further, these assumptions are based on the peak to peak stiffness definition which leads to a single stiffness value for every load cycle.

The experimental points are characterized by a low variability and they show a hyperbolic trend; they have been represented up to an experimental drift equal to the ultimate one. The ratio $k_{p-p},/k_{p-p,y}$ assumes values higher than 1 for $\theta/\theta_y <1$ since they represent column initial stiffness; however, the meaningful points are those for $\theta/\theta_y >1$ which represent the stiffness degradation in the post-elastic stage.

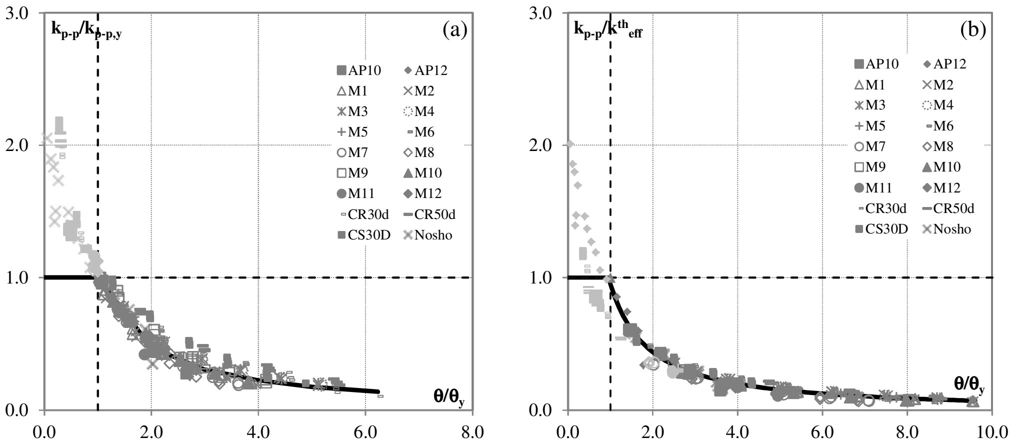


Figure 3. Experimental points and fitting curve for $k_{p-p},/k_{p-p,y}$ - θ/θ_y (a); for $k_{p-p},/k^{th}_{eff}$ - θ/θ^{h}_{y} (b).

Indeed, the experimental range for $\theta/\theta_y >1$ indicate the stiffness decrease once the yielding drift has been exceeded due to a seismic event; it is assumed that in the pre-yielding state the damage influence on the member stiffness is negligible . According to this assumption, the following simple theoretical expressions can be used to predict the stiffness degradation:

$$\lambda'_k = 1.0 \qquad \text{for} \qquad \theta/\theta_y \le 1.0 \tag{4.2}$$

$$\lambda'_k = 1.0 - \left(1.10 + 1.16 \cdot \left(\theta/\theta_y\right)^{-0.87}\right) \qquad \text{for} \qquad 1.0 < \theta/\theta_y \le \theta_u/\theta_y \tag{4.3}$$

The proposed expressions allow to estimate λ_k values for members on which it is possible to compute the drift level attained due to the seismic event. In order to use the above expressions the experimental yield rotation, θ_y, is needed. Therefore, in order to provide a suitable tool to be used for the theoretical assessment of the residual building capacity through pushover analyses on the structure in different damage state configurations, it is necessary to normalize k_{p-p} and θ values with respect to theoretical yielding stiffness and rotation, k^{th}_{eff} and θ^{h}_{y}, rather than experimental ones, $k_{p.p,y}$ and θ_y.
Theoretical yield rotation, θ^{th}_{y}, and stiffness, k^{th}_{eff}, can be computed as follows:

$$\theta^{th}_{y} = \frac{M_P L_S}{3EI_{eff}} \tag{4.4}$$

where M_p is the theoretical bending moment corresponding to bar yielding, and EI_{eff} is the effective member stiffness computed according to the expressions reported in (Elwood et al., 2007) (EI_{eff} = 0.3EI$_g$ for 0<ν≤0.1; EI_{eff} = 0.7EI$_g$ for ν>0.5; and EI_{eff} obtained using a linear interpolation in the range 0.1<ν≤0.5;with E=concrete modulus and I_g=moment of inertia of gross column cross-section).

$$k^{th}_{eff} = \frac{3EI_{eff}}{L_S} \tag{4.5}$$

The experimental points trend obtained by using those parameters to normalize stiffness and drift is reported in Figure 3 (b). The points scattering is very low and the trend is similar to that presented in Figure 3 (a). Since theoretical yield rotations are typically conservative with respect to the experimental ones, the experimental points are reported up a value θ/θ^{h}_{y} =10 (rather than θ/θ^{h}_{y} = 8, Figure 3 (a)). Based on the trend reported in Figure 3 (b), it is possible to determine the theoretical expressions which provide a best fitting of experimental data; in particular, the stiffness degradation modification factor $\lambda_k = k_{p\text{-}p},/k^{th}_{eff}$ can be calculated as a function of the θ/θ^{h}_{y} ratio as follows:

$$\lambda_k = 1.0 \qquad \text{for} \qquad \theta/\theta_y^{th} \le 1.0 \tag{4.6}$$

$$\lambda_k = 1 - \left(1.00 - 0.96 \cdot \left(\theta/\theta_y\right)^{-1.11}\right) \qquad \text{for} \qquad 1.0 < \theta/\theta_y^{th} \le \theta_u^{th}/\theta_y^{th} \tag{4.7}$$

The best fitting curve is shown in Figure 3(b) as continuous line.

5. STRENGTH MODIFICATION FACTOR

The strength modification factor is a measure of the member strength degradation after damage. In order to compute such factor, the experimental peak forces, F_i, have been determined for each test at different drift levels; these values have been normalized with respect to the maximum force for each test, $F_{max.}$, in order to make comparable the different test results. Both positive and negative peak and maximum forces have been determined and the relevant ratio $/F_i/ / /F_{max}/$ has been computed. The strength degradation is then determined as a function of the ratio $/\theta_i//\theta_y$; compared to the case of stiffness degradation, where for each cycle it was possible to define a single value for the peak-to-peak stiffness and associate it a single rotation value (average between the positive and the negative values), in the case of strength each peak corresponds to a single rotation and thus positive and negative values have been considered. Hence, the strength degradation modification factor, λ'_Q, has been defined as:

$$\lambda'_Q = \frac{|F_i|}{|F_{\max}|} \tag{5.1}$$

The experimental values of λ'_Q as a function of the ratio $/\theta_i//\theta_y$ are reported in Figure 4 (a).
The experimental points show a very similar trend up to$/\theta_i//\theta_y$ = 1, while a significant scattering can be observed for high values of $/\theta_i//\theta_y$; this can be explained considering that the experimental strength degradation may be significantly different for positive and negative horizontal load actions (i.e. the envelope cyclic experimental curves are often not symmetrical due to the damage initiation in one direction); further, number and yield strength of columns longitudinal rebars could significant influence the strength drop after the maximum force has been experienced. In spite of the large scatter, a regression formulation is still proposed; in fact the strength degradation is in any case limited by the attainment of θ_u (that is set at the 20% drop of F_{max}).

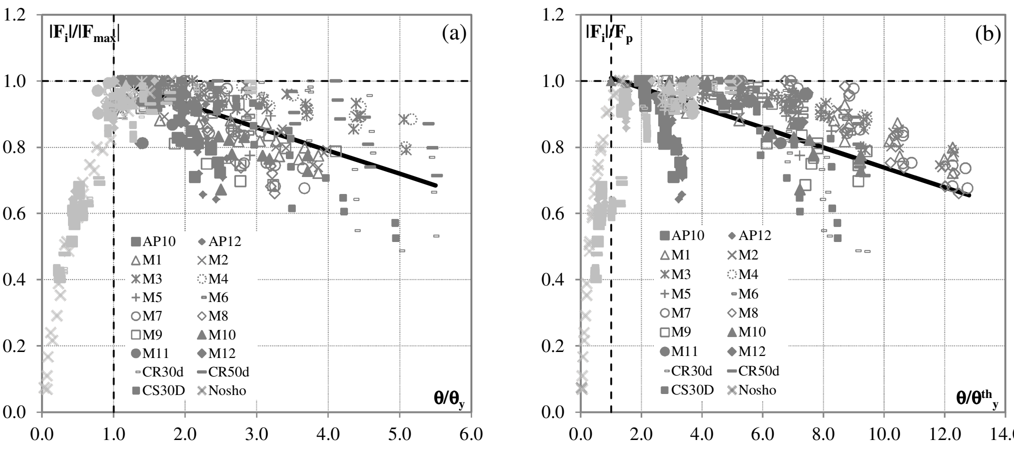


Figure 4. Experimental points and fitting curve for $/F_i/ / /F_{max}/$ - $/\theta_i//\theta_y$ (a); for $|F_i|/F_P$ - $|\theta_i|/\theta_y$ (b).

The degradation theoretical law obtained as a best fitting of experimental results is also depicted in Figure 4 (a). The strength degradation obviously starts when the maximum lateral force is experienced; this value is typically attained for $|\theta_i|/\theta_y$ ratio equal to about 1. Based on this assumption, the interpolating curve has been determined for the assessment of λ_Q starting from $\theta/\theta_y > 1$ while a constant value $\lambda_Q = 1$ has been assumed for $\theta/\theta_y < 1$. Note that in the regression formula the notation θ is introduced, instead of $|\theta_i|$, in order to represent the generic rotation demand, not necessarily associated to a cyclic test:

$$\lambda'_Q = 1.0 \qquad \text{for} \qquad \theta/\theta_y \le 1.0 \tag{5.2}$$

$$\lambda'_Q = 1.0 - 0.07 \cdot (\theta/\theta_y - 1.0) \qquad \text{for} \qquad 1.0 < \theta/\theta_y \le \theta_u/\theta_y \tag{5.3}$$

A linear relationship for degradation law has been chosen.

To overcome the difficulties related to the computation of θ_y as well as of the experimental maximum lateral force which can be sustained by the column, the experimental trends have been also computed by using the theoretical parameters F_p and θ^{th}_y representing the maximum theoretical force and yield rotation respectively. In particular, F_p, can be computed as M_p/L_S while expression (4.5) has been used for θ^{th}_y. By using these parameters to normalize the maximum force and the member drift, the trend reported in Figure 4 (b) is obtained.

Based on the trend reported in Figure 4 (b), it is possible to determine the theoretical expression which provide a best fitting of experimental data; in particular the strength degradation modification factor $\lambda'_Q = |F_i|/F_P$ can be calculated as a function of the θ/θ^{th}_y ratio as follows:

$$\lambda_Q = 1.0 \qquad \text{for} \qquad \theta/\theta_y^{th} \le 1.3 \tag{5.4}$$

$$\lambda_Q = 1.0 - 0.03 \cdot (\theta/\theta_y^{th} - 1.3) \qquad \text{for} \qquad 1.3 < \theta/\theta_y^{th} \le \theta_u^{th}/\theta_y^{th} \tag{5.5}$$

In this case the strength degradation begins for $\theta/\theta^{th}_y > 1.3$ due to the conservative values provided by theoretical expression in the computation of the yield drift.

6. RESIDUAL DRIFT MODIFICATION FACTOR

During earthquake ground motion, plastic deformation of materials involves residual deformation of structural elements, particularly plastic rotations. The residual drift, *RD,* is here defined as the plastic rotation measured on the column for an external lateral load equal to zero. As previously reported for the member stiffness and strength, Figure 5 (a) depicts the experimental residual drift attained on columns at different drift levels. In particular, *RD* has been normalized with respect to the yield rotation as well as the drift in order to compare the data resulting from the database. *RD* an θ values reported in Figure 5 (a) are obtained as average values recorded for positive and negative external load actions. Figure 5 (a) shows that an ascending pseudo-parabolic trend of *RD* is attained for increasing values of θ/θ_y ratios. This confirms that *RD* increases for actions overcoming the member elastic threshold and are clearly negligible for $\theta/\theta_y<1$. Thus to provide a suitable analytical expression for RD/θ_y prediction as a function of θ/θ_y, an interpolating curve has been considered for $\theta/\theta_y>1$ only (Figure 5 (a)):

$$RD/\theta_y = 0.0 \qquad \text{for} \qquad \theta/\theta_y \le 1.0 \tag{6.1}$$

$$RD/\theta_y = 0.02(\theta/\theta_y - 1)^2 + 0.48(\theta/\theta_y - 1) \qquad \text{for} \qquad 1.0 \le \theta/\theta_y \le \theta_u/\theta_y \tag{6.2}$$

As for stiffness and strength also in this case it is possible to normalize both *RD* and θ with respect to θ^{th}_y rather than θ_y; in this way it is possible to provide a proper modification factor of member plastic hinge in the pushover analysis according to theoretical provisions. The experimental points trend obtained in such a case is reported in Figure 5 (b). The corresponding interpolating best fitting curve is depicted in Figure 5 (b)**.** Note that since θ^{th}_y is typically lower than that recorded in the experiments, the range variation of ratio rather than θ/θ^{h}_y is significantly wider than that related to θ/θ_y. The best fitting curve analytical expressions are:

$$RD/\theta_y^{th} = 0.0 \qquad \text{for} \qquad \theta/\theta_y^{th} \le 0.8 \tag{6.3}$$

$$RD/\theta_y^{th} = 0.05\left(\theta/\theta_y^{th} - 0.8\right)^2 + 0.09\left(\theta/\theta_y^{th} - 0.8\right) \qquad \text{for} \qquad 0.8 < \theta/\theta_y^{th} \le \theta_u^{th}/\theta_y^{th} \tag{6.4}$$

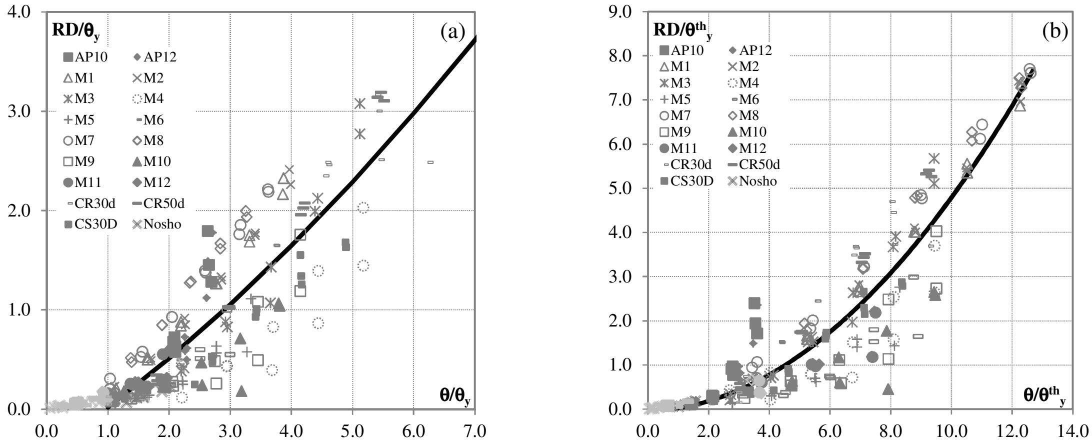


Figure 5. Experimental points and fitting curve for RD/θ_y - θ/θ_y (a); for RD/θ^{h}_y - θ/θ^{h}_y (b).

Thus for each column cross section, once the θ^{h}_y has been computed, it possible to easily determine *RD* for a given drift level provided by an earthquake. In this way the plastic hinge can be modified by simply reduce the plastic deformation capacity.

6. CONCLUSIONS

A database consisting of 18 tests results on flexure or flexure-shear controlled square/rectangular RC columns reinforced with deformed rebars under constant axial load and cyclic horizontal actions has been assembled. The experimental outcomes in terms of lateral load versus drift have been used to calibrate modification factors for plastic hinges of damaged columns representative of under-designed columns typical of Mediterranean European regions existing buildings. In particular, suitable stiffness, strength and residual drift modification factors have been proposed as a function of member drift in order to account for structural members damaging due to earthquake ground motions when non-linear analysis are performed. The modification factors have been obtained referring to mechanical parameters (yield stiffness and rotation) provided by experimental tests or theoretical expressions. The modification factors can be used to easily determine the structural seismic capacity of a damaged structure. For a given damage level (observed by visual in situ inspections or theoretically assumed) they allow to proper modify the plastic hinge properties in the nonlinear pushover analysis and thus to quantify the structural deformation capacity decrease induced by the damage. The procedure could help engineers in the assessment of structure safety loss of damaged buildings and, in the case of acceptable safety decay, in the choice of the most appropriate method for their repair and/or strengthening.

In order to derive more reliable modification factors, further tests are necessary to enrich the existing database; furthermore an extension of the procedure on RC columns reinforced with smooth bars, widely used in the past years in Mediterranean European regions, is under investigation.

AKCNOWLEDGEMENT
This study was performed in the framework of PE 2010–2013; joint program DPC-Reluis Task 1.1.2: Ordinary and Precast Reinforced Concrete Structures.

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Table 1. Geometrical and mechanical parameters for selected tests

Reference	(Atalay and Penzien, 1975)		(Nosho et al. 1996)	(Matamoros et al., 1999)												(Di Ludovico et al., 2009)		
Specimen	No.10	No.12	No.1	C10-05N	C10-05S	C10-10N	C10-10S	C10-20N	C10-20S	C5-00N	C5-00S	C5-20N	C5-20S	C5-40N	C5-40S	R300D_c	R500d_c	S300d_c
Label	AP10	AP12	Nosho	M1	M2	M3	M4	M5	M6	M7	M8	M9	M10	M11	M12	cr30d	cr50d	cs30d
b [mm]	305	305	279.4	203	203	203	203	203	203	203	203	203	203	203	203	500	300	300
h [mm]	305	305	279.4	203	203	203	203	203	203	203	203	203	203	203	203	300	500	300
L_s [mm]	1676	1676	2134	610	610	610	610	610	610	610	610	610	610	610	610	1500	1500	1500
ρ_t [%]	0.37	0.37	0.10	0.92	0.92	0.92	0.90	0.92	0.90	0.92	0.90	0.92	0.90	0.92	0.90	0.13	0.22	0.22
ρ_l [%]	1.63	1.63	1.02	1.93	1.93	1.93	1.93	1.93	1.93	1.93	1.93	1.93	1.93	1.93	1.93	0.90	0.57	1.00
ν	0.27	0.27	0.34	0.05	0.05	0.10	0.10	0.21	0.21	0.00	0.00	0.14	0.14	0.36	0.36	0.12	0.10	0.18
s/h	0.47	0.47	0.90	0.45	0.45	0.41	0.42	0.41	0.43	0.40	0.41	0.45	0.46	0.41	0.42	0.54	0.31	0.54
f_{cm} [Mpa]	32.4	31.8	40.6	69.6	69.6	67.8	67.8	65.5	65.5	37.9	37.9	48.3	48.3	38.1	38.1	18.85	18.85	18.85
f_{yl} [Mpa]	363.0	363.0	407.0	586.1	586.1	572.3	573.3	572.3	573.3	572.3	573.3	586.1	587.1	572.3	573.3	520.0	520.0	520.0
f_{yt} [Mpa]	392.0	373.0	351.0	406.8	406.8	513.7	514.7	513.7	514.7	513.7	514.7	406.8	407.8	513.7	514.7	520.0	520.0	520.0
F^+_{max} [kN]	90.2	93.5	69.7	73.2	70.8	103.7	100.9	123.7	118.3	59.2	58.3	79.2	76.0	97.0	96.5	91.5	144.8	61.4
F^-_{max} [kN]	-90.2	-91.4	-58.1	-69.3	-69.2	-99.7	-100.0	-118.2	-121.0	-57.4	-56.5	-77.9	-78.0	-94.3	-91.8	-98.8	-144.7	-72.5
θ^+_{Fmax} [%]	1.62	1.92	1.48	2.07	1.94	2.82	2.79	2.82	3.12	3.06	3.08	2.80	2.77	2.82	2.82	2.18	1.96	3.05
θ^-_{Fmax} [%]	-1.82	-1.71	-1.36	-2.12	-2.05	-2.99	-2.92	-3.03	-3.13	-3.11	-2.92	-1.90	-1.93	-2.97	-2.92	-2.81	-2.76	-2.25
θ^+_y [%]	1.11	1.22	0.82	1.90	1.83	1.65	1.65	2.33	2.21	2.21	2.25	2.20	1.65	2.25	2.09	1.26	0.80	1.58
θ^-_y [%]	-1.19	-1.09	-0.92	-2.00	-1.86	-1.42	-1.41	-1.88	-2.14	-2.04	-2.29	-1.53	-1.91	-2.67	-2.65	-1.03	-0.67	-1.14
θ^+_u [%]	2.85	2.82	1.67	6.84	7.28	7.87	7.93	7.11	7.38	6.69	6.69	5.48	5.46	5.61	5.33	5.47	3.65	5.47
θ^-_u [%]	-2.95	-2.91	-1.62	-7.30	-7.07	-8.20	-8.36	-7.00	-7.34	-6.95	-6.84	-6.00	-5.49	-5.21	-5.07	-3.87	-3.68	-3.87
k_{Elw} [$kNmm^{-1}$]	4.91	4.88	3.27	7.28	7.62	12.03	12.10	11.28	10.10	6.03	5.80	8.68	8.80	8.61	8.84	5.55	12.96	3.37
k_y [$kNmm^{-1}$]	4.34	4.37	2.99	5.94	6.15	9.97	9.92	8.86	8.53	4.37	4.06	6.62	6.84	6.34	6.48	4.61	9.98	2.72