MACRO-MODELS FOR THE IN-PLANE ANALYSIS OF INFILLED RC FRAMES: AN APPRAISAL THROUGH PSEUDO-DYNAMIC TESTS

Felice COLANGELO[1]

SUMMARY

The paper is intended to evaluate the effectiveness of existing macro-models in simulating inelastic behaviour of brick-infilled RC frames under earthquake loading. Structural members are idealised as linear beams provided with hysteretic springs; filler wall is modelled as crushing struts bracing the frame. Model validation is based on comparing numerical seismic response in global terms to time-histories from pseudo-dynamic tests upon one-floor one-bay half-scale specimens. In addition, ability in representing sustained structural damage is investigated. A number of indices here expressed in terms of both measured and computed plastic rotations are correlated to damage observed on frames attaining to complete failure during cyclic tests following the pseudo-dynamic ones. Substantially, one concludes that macro-models can capture the experimental behaviour. Nevertheless, confidence in quantifying parameters is not allowed.

INTRODUCTION

Nowadays, inelastic dynamic analysis of infilled framed structures subjected to severe base motion is commonly performed in the field of code issuing for new buildings (e.g., where irregularity is of concern) as well as in performance assessment for existing under-designed buildings. Such computation requires very efficient models in order to handle constructions of usual complexity. As a matter of fact, macro-models alone make feasible seismic analyses in three dimensions of actual buildings beyond the elastic stage. Although significant steps towards realistic modelling of filler panel have been taken with the formulation of quite accurate behavioural laws and with the (simplified) coupling of the in-plane and out-of-plane response [Fardis 1997], studies cannot be considered as exhaustive. That mostly holds as concerns model calibration and validation. In fact, as someone said, macro-model ability consists of representing rather than predicting possible phenomena, in a pragmatic and macroscopic fashion. Sometimes numerical parameters are not even related to a clear or unique physical meaning, as in the case where filler walls present openings. Moreover, if it deals with non-structural brick masonry made up of poor and uncontrolled materials, then mechanical properties result as scattered and unknown to a large extent. It follows that model calibration often becomes a major, difficult task.

Both finite-element analysis and laboratory testing have been resorted to in order to calibrate and validate macro-models. However, since detailed numerical investigation is limited to monotonic analysis, it merely suggests an optimistic envelope for seismic response. On the other hand, empirical calibration mostly relies on cyclic tests, aiming at results of a clear interpretation. In comparison, few works appraised the effectiveness of macro-models in tracing an infilled frame response under random cycling. The paper is intended to contribute to such a matter by examining existing macro-models. Pseudo-dynamic (PD) experiments furnish reference data for validating numerical time-histories and structural damage representation according to a number of indices proposed in the literature. Being the effect of PD tests upon frame members light at sight, cyclic tests were performed until bare specimens clearly failed, so damage indices themselves are validated on the experimental basis.

[1] *Dept of Structural, Hydraulic and Geotechnical Engineering, University of L'Aquila, ItalyEmail: col@ing.univaq.it*

EXPERIMENTAL ACTIVITY

Six half-scale brick-infilled RC frames were tested (**Figure 1**). Specimens named C and L were designed according to the Eurocode 8, whereas specimens N comply with the earlier Italian seismic code prescribing allowable stress verification. In any case, columns exhibit over-strength compared to beams. With reference to specimens N, reinforcing steel class is lower and detailing appears as poor. In fact, stirrups, presenting a larger spacing, are not even anchored to the concrete core, and lapped splices are placed at column base. Unreinforced masonry was built up according to the Italian practice, that is in contact with RC members with neither joints nor connectors. Mortar and vertical-hole bricks arranged in a single wythe were used for specimens C and L, horizontal-hole bricks forming two unconnected skins for specimens N. The study does not cover openings.

As concerns experiments, subsequently to initial elastic measurements a PD test was carried out with infilled specimens adopting the Tolmezzo (Friuli) 5/6/76 E-W accelerogram. Constant vertical loads were maintained on beam-column joints to simulate column axial force due to gravity action. The effect on frame of that test merely consisting of permanent cracks at the beam ends and along columns, the same experiment was repeated with two bare specimens and leaving infilled the other ones. No repair work was undertaken in the meanwhile. Due to the repeated PD test, specimen N1 suffered some cover spalling and specimen N2 just visible rebar buckling, both located in the beam critical regions. Such phenomena occurred to a large extent in all the specimens during final cyclic tests performed with bare frames until they clearly failed. Contrary to RC members, already starting from the first PD test filler walls sustained heavy damage, consisting of centre crushing in specimens C, corner crushing in specimens N and corner crushing prevailing on joint sliding in specimens L.

Table 1 and **Table 2** summarise salient results of PD tests. Frame displacement ductility derives from the model described in the next section. Wall ductility demand conservatively refers to panel first cracking as the yield drift, actually registered quite early and denoting a very brittle behaviour (**Table 1**). Briefly, results confirm that filler walls play an essential role in defining building stiffness and strength, even if frames suitable for withstanding seismic action and unreinforced hollow brick masonry are of concern. The infill strengthening effect remains noticeable until the repeated PD test (**Table 2**), in spite of the extensive observed damage that displacement ductility demand and stiffness decrease testify. An exhaustive description of the experimental outcome has been presented elsewhere, together with mechanical property qualification for materials and specimens as well [Colangelo 1999; Nuti, Biondi and Colangelo 1998].

GLOBAL MODELS OF THE IN-PLANE BEHAVIOUR

Attention is paid to well-known phenomenological models widely used for inelastic dynamic analyses and

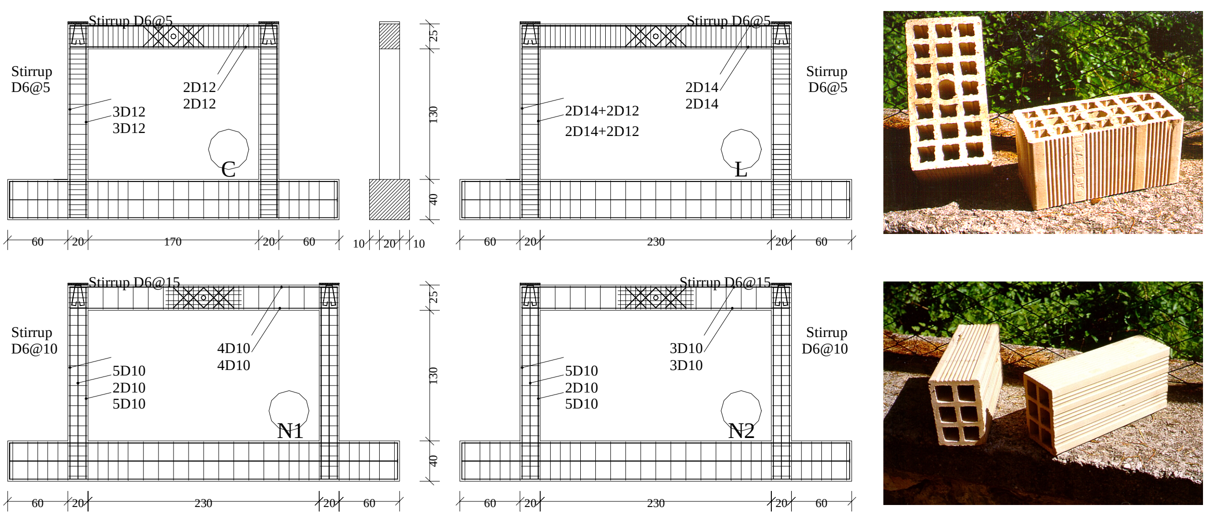

Figure 1: RC specimens (two samples each one for types named C and L) with corresponding bricks

Table 1: Results from PD tests upon virgin specimens

Specimen	C1	C2	L1	L2	N1	N2
Initial period of four-storey prototype building	0.12s	0.13s	0.15s	0.14s	0.14s	0.14s
Infilled frame stiffness to bare frame stiffness	9.6	8.2	7.7	8.9	10.3	9.4
First cracking drift	0.29‰	0.22‰	0.26‰	0.25‰	-	0.25‰
Cracked stage stiffness to bare frame stiffness	2.8	2.0	1.8	2.3	2.2	2.2
Infilled frame reaction to bare frame strength	1.7	1.8	1.6	1.7	1.9	1.8
Frame displacem. ductility (demand / capacity)	0.8 / 5.7	1.5 / 5.4	2.0 / 5.0	1.9 / 5.1	3.4 / 8.4	2.6 / 6.1
Wall displacement ductility (demand)	18.3	48.4	62.9	64.9	-	77.5
Maximum acceleration to ground acceleration	1.7	1.9	1.4	1.5	1.0	1.4
Final stiffness to initial stiffness	16%	9%	5%	5%	3%	4%

Table 2: Results from repeated PD tests upon damaged specimens

Specimen	C1	C2(bare)	L1(bare)	L2	N1	N2
Initial period of four-storey prototype building	0.31s	0.68s	0.82s	0.60s	0.82s	0.62s
Infilled frame reaction to bare frame strength	1.9	1.0	1.0	1.5	1.7	1.5
Frame displacem. ductility (demand / capacity)	2.0 / 5.7	3.5 / 5.4	2.9 / 5.0	2.7 / 5.1	3.3 / 8.4	2.9 / 6.1
Wall displacement ductility (demand)	49.5	-	-	92.9	-	85.3
Maximum acceleration to ground acceleration	1.9	1.0	0.9	1.3	0.9	1.1
Final stiffness to initial stiffness	34%	53%	72%	48%	64%	49%

whose parameter quantification can rely on directives suggested in the literature. Following the usual simplified approach of lumped plasticity, RC members are modelled as linear beam elements provided with point hysteretic springs at their ends. The Takeda model as modified by Otani and Litton is adopted to represent the moment-rotation law. Despite gross simplifications in neglecting interaction between bending moment and axial force, in assuming anti-symmetric member flexure, in disregarding shear failure and strength degradation as well, such a model is still used in extensive parametric analyses [Fardis 1997]. This is due to its simplicity and spreading.

As regards filler panel, two behavioural laws are considered in conjunction with equivalent diagonal struts. The first one has been presented in [Panagiotakos and Fardis 1994]; it assumes a piecewise linear skeleton curve with corners corresponding to cracking, peak strength and failure. The envelope curve falls down exponentially as inelastic cycles proceed, providing strength degradation. Pinching is represented too. In addition, the study treats of the pioneer model proposed in [Klingner and Bertero 1978], the purpose being to verify to what extent such simplifications as lacking of cracking, pinching and envelope curve degradation invalidate simulation. Therefore, equal values are assumed for parameters related to the same phenomena. In particular, the initial stiffness of the latter model corresponds to the cracked-wall actual stiffness, so the peak-strength displacement results as quite larger with respect to the former, providing also shear stiffness instead. On the other hand, the post-peak exponentially falling branch of the latter model presents the same secant slope as the linearly falling branch of the first one. Parameter quantification was discussed in some detail elsewhere, where suggestions available in the literature were recalled and calibration for present specimens was presented [Colangelo 1998]. Actually, some parameters were tuned empirically instead of being anticipated, mainly those ones related to cyclic degradation. Nevertheless, a certain effort was made to retain parameter definition as close as possible to suggested directives. In addition, a simple criterion of calibration was identified as reasonably valid for all the specimens.

Global seismic response

The aforesaid models are appraised in representing PD global response of specimens in this section. **Figure 2** allows one to compare experimental and numerical time-histories relevant to specimen L2. A similar comparison extended to all the specimens supports the following remarks. First, the model by Klingner and Bertero considerably overestimates displacement in the earlier stage of the first PD test, clearly due to neglecting shear behaviour of the wall. Nevertheless, maybe unexpectedly, peak displacements are similar to those obtained by using the Panagiotakos and Fardis model, furthermore they agree with the experimental ones. The model by Panagiotakos and Fardis again works better in post-peak oscillations, it is believed thanks to taking account of pinching. On the other hand, agreement with measured tails of oscillation appears as poor for both models. Coming to storey shear, the main result is that strength degradation becomes apparent only in the course of the repeated PD test, where a lower reaction corresponds to larger displacements compared to the previous test. The Panagiotakos and Fardis model alone is able to capture with accuracy such an aspect that, however, would not seem so important in the case of few oscillations. Finally, as far as hysteresis dissipated energy is concerned, it is worth mentioning that the model by Klingner and Bertero leads to systematic overestimation, mostly at the end of the repeated PD test. The reason is that larger or similar displacements, compared to the experimental ones and to those predicted by Panagiotakos and Fardis, combine with excessive strength, as discussed above. Frame and wall contributions to energy dissipation are almost equal, specimens C making the exception.

Figure 3 shows time-histories of specimen C1, the experimental ones versus those obtained with the model by Panagiotakos and Fardis only. Dotted curves derive from assuming parameter values according to the common criterion mentioned previously: agreement between experimental and simulated time-histories is indeed poor. Actually, the same criterion leads to very satisfactory representations for the twin specimen C2. Quite evident differences between responses of specimens C are noted in **Table 1** too. Well, **Figure 3** reports also numerical time-histories obtained by increasing wall strength and cracked-stage stiffness of 40%, a quantity consistent with measure scattering in compressive tests carried out upon wallettes, and by decreasing to some extent the envelop curve deterioration rate. Time-histories almost overlap now. Such result equally highlights the very strong influence of filler walls on the seismic response, the consequent sensitivity of numerical simulations to wall mechanical properties and, unfortunately, to their scattering too, finally the necessity of special qualification of materials and model calibration in order to avoid inelastic dynamic analyses being really misleading.

Local structural damage

This section focuses on the local analysis of sustained structural damage. First, visible damage as observed on RC members subsequent to experiments has been classified depending on severity (**Table 3**). It is worth distinguishing columns from beams since full-dept flexural cracks characterised damage in the latter only, so the same phenomenon (e.g. cover spalling) seems more severe. Furthermore, concrete crushing was exclusive for columns due to applied vertical loads. Damage resulting at the end of each test or as soon as a rebar fractured

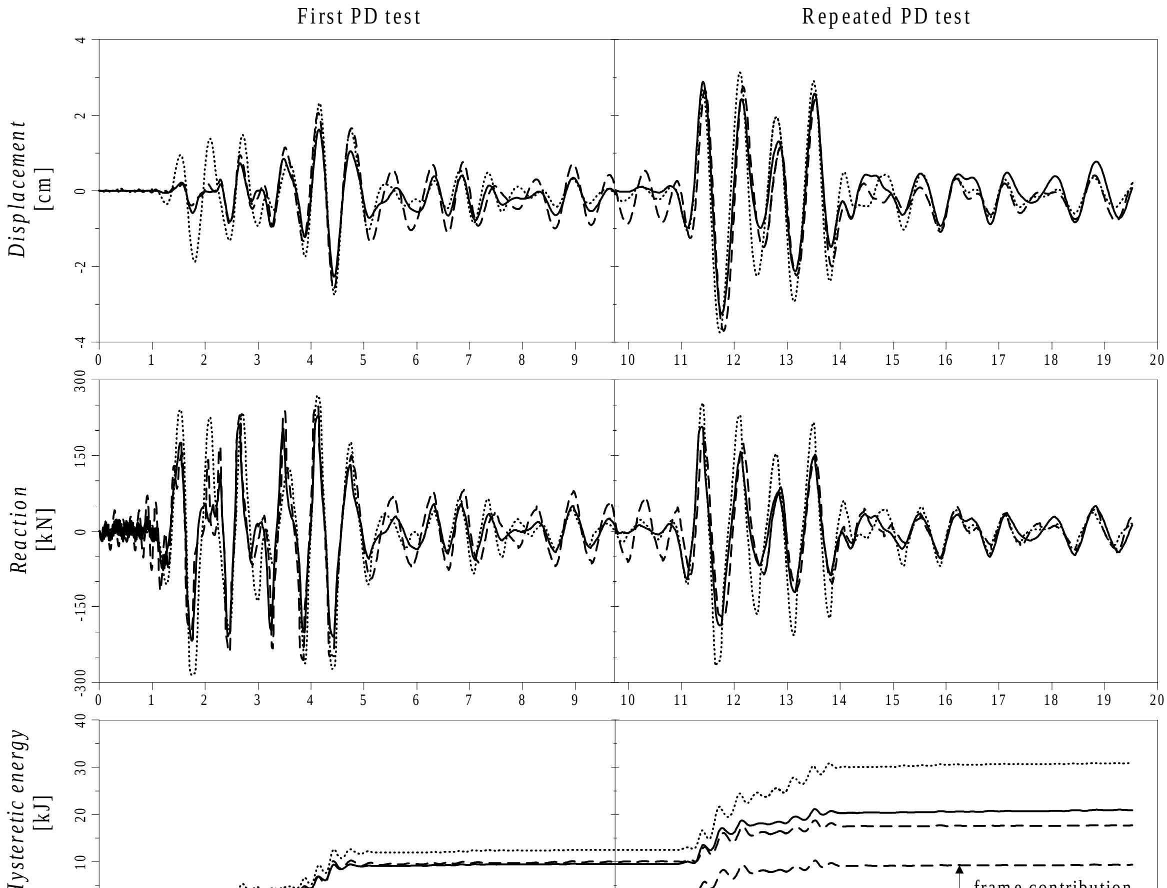

Figure 2: Time-histories of specimen L2 (—— PD test; simulation according to Klingner and Bertero; - - - - simulation according to Panagiotakos and Fardis)

first, if any, during cyclic tests is used to validate macro-models together with various damage indices proposed in the literature. In detail, the investigation regards the kinematic index, the cyclic index, the hysteretic index, the plastic-fatigue index by [Stephens and Yao 1986] in its simplified form, the combined index by [Park and Ang 1985], the combined index by [Banon and Veneziano 1982]. Parameter values recommended in the literature are adopted. Indices are expressed in terms of both experimental and numerical rotations of member critical regions; with reference to those ones computed from measured rotations, it should be noted that bending moment values in member sections are not available from tests upon frames. Therefore, the model by Clough and Johnston has been adopted to rebuild time-histories of the moment-rotation law, given the sequence of measured rotations.

In order to obtain consistent quantities varying from zero in the elastic stage to one at failure, damage indices are derived as an equivalent ductility demand divided by the equivalent ductility capacity resulting from a monotonic test to failure. Unfortunately, such a normalisation involves the difficult and questionable matter of defining collapse, reflecting on the failure rotation assumed for computation. Three methods giving failure and yield rotation as well have been evaluated: the semi-empirical formulation by [Park and Ang 1985], the analogous one by [Park, Ang and Wen 1987], the last formulation (MC in the follows) consisting of strip analyses of RC section under monotonic bending moment and constant axial force. Strip analyses conservatively consider failure curvature as achieved as soon as steel strain attains the peak-stress elongation, or core extreme fibres reach the failure confined compressive strain, whichever occurs first. Then curvature ductility is turned into rotational ductility by adopting the plastic hinge length proposed by Mattock.

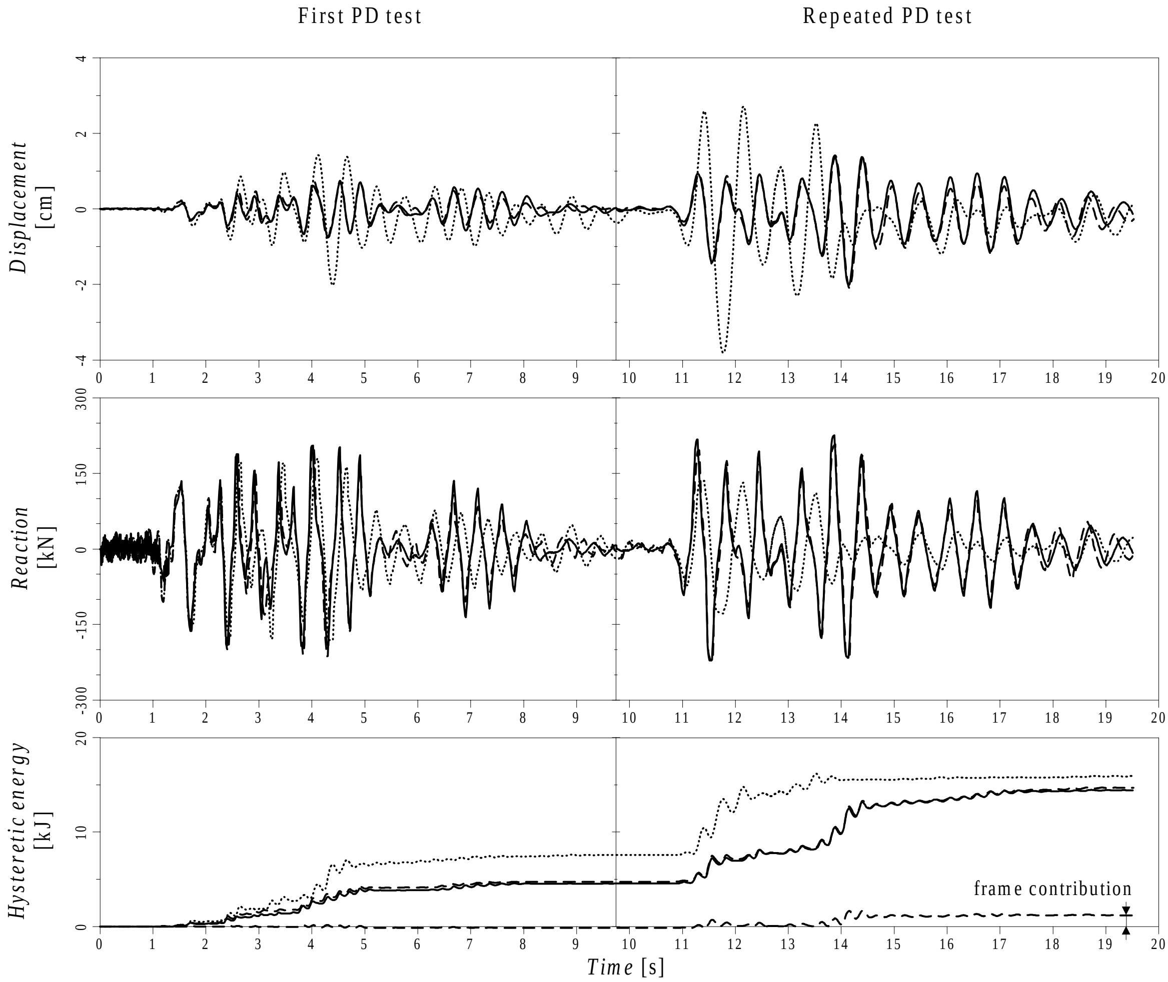

Figure 3: Time-histories of specimen C1 (—— PD test; simulation according to Panagiotakos and Fardis; - - - - simulation according to Panagiotakos and Fardis, wall parameters tuned)

Table 3: Classification of observed damage

	Notation	Damage description
Column	K	Permanent cracking of concrete.
	H and HH	Concrete crushing, exterior (H) or as deep as to make stirrups visible (HH).
	S and SS	Cover spalling with longitudinal rebar made just visible (S) or bare about 20cm long (SS).
	B	Visible buckling of longitudinal rebar.
Beam	KK	Permanent full-dept cracking of concrete.
	SS	Cover spalling with longitudinal rebar made visible.
	BB	Visible buckling of longitudinal rebar.
	F	Longitudinal rebar fracture (first occurrence).

Coming to results, **Figure 4** reports index values computed with measured rotations. Each graph collects all the indices for the same critical region on the right-hand side of specimens, at the end of that test and using that yield and failure formulation specified on the ordinate axis. The upper horizontal axis of every graph reports visible damage according to the classification in **Table 3**, relevant to the corresponding specimen marked on the abscissa axis. Starting from the left-hand graphs, one can appreciate the accuracy in predicting failure by comparing visible damage and indices at the end of cyclic tests. Independently of index used, the formulation by Park and Ang (top graph) leads to a considerable underestimation of beam damage: no value exceeds 0.7, despite observed buckling and fracture of rebar. Instead, agreement might be considered as satisfactory for columns: various criteria capture buckling for specimen N1, even if extensive spalling in specimens L2 and N2 appears as underestimated. Overall, the formulation fails, because it does not advise of the major damage sustained by beams compared to columns. The same remark holds if one looks at results from the formulation by Park, Ang and Wen, not displayed for brevity, though inconsistency between beams and columns attenuates to some extent and more severe damage is predicted. The MC formulation (bottom graph) captures best comparative damage sustained by beams and columns, with the exception of specimen N1. From the point of view of quantity, except for those by the cyclic criterion and by Stephens and Yao, values appear as excessive, not so much for the beams, which certainly collapsed, as for the columns. The reason might lie in the conservative definition assumed for failure. Graphs on the right-hand side of **Figure 4**, pertinent to PD tests, confirm that damage is somewhat overestimated for beams, the two aforementioned indices making the exception. As regards columns, in the repeated PD test more pronounced damage is correctly obtained for specimens C2 and L1, tested bare. Finally, the index by Stephens and Yao is recognised as in the best agreement with experimental evidence, since in cyclic tests the cyclic index, remaining well below the value of one, does not capture beam failure.

With reference to indices derived from numerical rotations, conclusions drawn above about both qualitative and quantitative aspects are confirmed. In addition, it can be noticed that on the whole beam damage results slighter and column damage heavier, so that a more balanced situation appears. **Figure 5** shows values obtained with the MC formulation. Some differences between results given by Klingner and Bertero, on the left-hand side, and

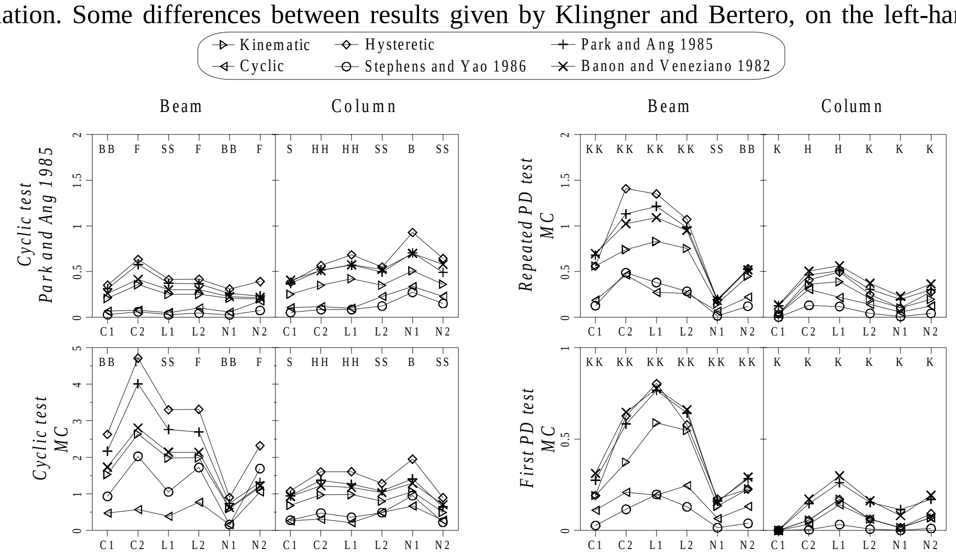

Figure 4: Observed damage versus damage indices according to measured rotations

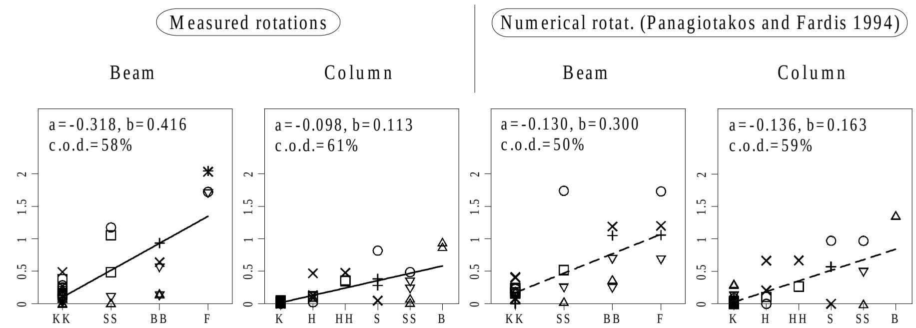

Figure 6: Correlation of observed damage with damage index (Stephens and Yao 1986, MC)

those by Panagiotakos and Fardis, on the right-hand side, are noted in PD tests (bottom graphs). The latter model reflects the lower damage that experimental measures (**Table 1** and **Figure 4**) suggest for members of specimen C2 and, mostly, C1. Unfortunately, visible damage, just cracking, does not allow validating such a distinction. This discrepancy has a negligible effect on the succeeding cyclic tests upon bare specimens (top graph).

In conclusion, it is worth observing that indices in both **Figure 4** and **Figure 5** appear as similar in their trend,

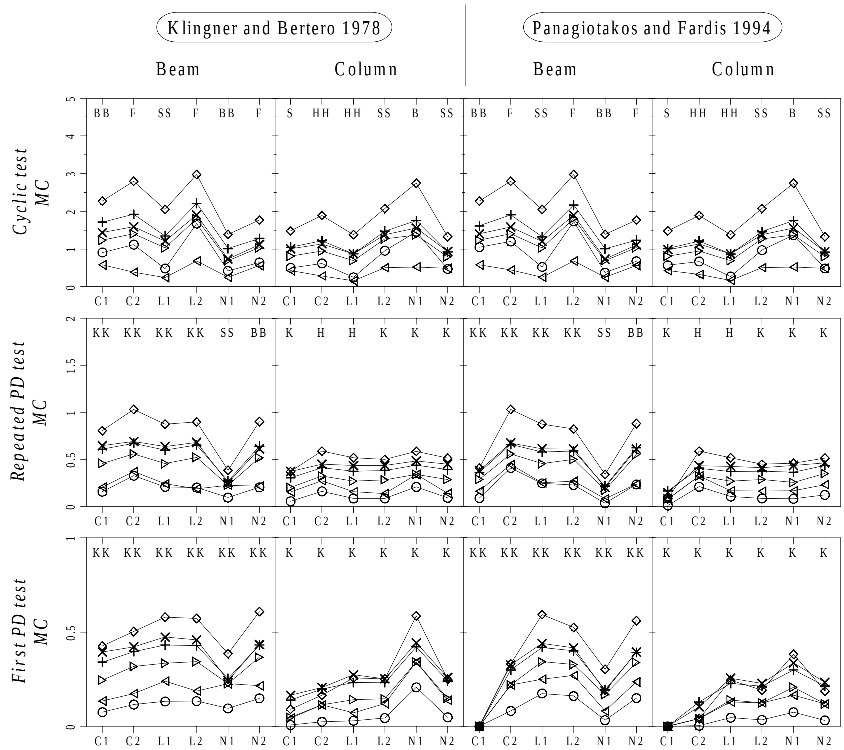

Figure 5: Observed damage versus damage indices according to numerical rotations

though they denote different damage severity inasmuch as scaled differently. If one looks at the Stephens and Yao criterion under the MC formulation, a reasonable agreement is noticed between observed and assessed damage. However, a clear underestimation occurs for the beam N1. Actually, the error appears as inherent in all the indices considered. The reason is related to the failure rotation appraisal, since rotational ductility demand results high for beams N in comparison with the other ones. In addition, strip analyses did not account for rebar buckling and it can be recognised that poor detailing penalised beams more than columns due to axial force. **Figure 6** displays the correlation between visible damage, assumed as linearly increasing, and the damage index of Stephens and Yao in conjunction with the MC formulation. Although the large dispersal allows little more than a comparison in statistical terms, graphs make evident how measured and numerical rotations present almost the same situation. One should also note some underestimating of damage in beams and overestimating in columns from computed rotations, as reported above.

CONCLUSIONS

Macro-models prove able to reproduce response quantities of interest in inelastic dynamic analyses of infilled frames. It is worth noting that even a simple model can capture peak displacements, it must be said retaining parameters their physical meaning. Obviously, taking account of the wall shear behaviour, pinching and strength degradation yields quite accurate results. Local analysis demonstrates at least that numerical lumped-plasticity rotations are consistent with measured ones, in part thanks to agreement between displacements, therefore structural damage assessment can be performed. Nevertheless, parameter quantification for macro-models and damage index as well, once the appropriate criterion has been selected, really reveals as crucial in order to obtain meaningful results. Wall property scattering, rotational capacity uncertainty and difficulty in quantifying damage complicate such a task to a large extent. Moreover, it should be noted that the study does not cover failure modes different from frame flexural mechanism and wall crushing. Within this ambit, specimens considered here are believed to represent a significant sample as regards buildings suitable for withstanding seismic action.

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