Paper No. 1360. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

PSEUDODYNAMIC TESTS ON BRICK-INFILLED R.C. FRAMES

F. COLANGELO

D.I.S.A.T. Department, University of L'Aquila,
Monteluco di Roio, 67040 L'Aquila (Italy)

ABSTRACT

The pseudodynamic testing facilities which have been set up in the D.I.S.A.T. Dept. laboratory, University of L'Aquila, are briefly described. Results are reported concerning a pilot test carried out on a half-scaled, one-floor one-bay, brick-infilled R.C. frame. The experimental results are compared with the numerical ones given by the hysteretic model proposed by Klingner and Bertero for infills.

KEYWORDS

Pseudodynamic test; infilled frame; partition; equivalent strut.

INTRODUCTION

As well known, the response of framed buildings subjected to earthquake action is significantly affected by the presence of infill walls (Brokken and Bertero, 1981). Amongst their favorable effects, it is worth mentioning the strength and stiffness increase, as well as the ability in dissipating more input energy. However, various examples of bad structural behavior have been observed, due also to ordinary partitions, such as torsion because of their asymmetrical plan disposition or failure, soft-story mechanism, short-column formation owing to partial infilling of the interstory, localized crushing of structural members interacting with walls. It is clear that one cannot say to neglect infills in favor of safety.

At present, it is hard to predict with a satisfactory degree of reliability how infilled r.c. frames would behave under strong shaking, even if refined, powerful tools are available to perform structural analyses. That holds because various uncertainties affect the definition of both mechanical properties and idealized behavior itself. In fact, the former ones are usually variable in a wide range for the materials involved, the latter should foresee different possible modes of failure, depending on parameters which appear as numerous (span-to-height ratio, wall-to-frame stiffness, beam-to-column stiffness, position and dimension of openings, frame-infill connection, presence of vertical loads), as well as difficult to quantify (human skill and care).

Lacking now both reliable and accessible methods of analysis, European normative codes (Eurocode 8, 1994*b*) mostly report qualitative regulations to account for infills in ordinary design. Research focuses attention on laboratory testing and numerical simulation as well (Calvi *et al.*, 1993). Concerning the aforementioned tasks, the pseudodynamic test method (Mahin *et al.*, 1989) appears to be an effective technique in pursuing such aims as understanding modes of failure by means of laboratory experiments, model verification and numerical parameter calibration.

PSEUDODYNAMIC EQUIPMENT

The pseudodynamic testing facilities which have been set up in the D.I.S.A.T. Dept. laboratory, University of L'Aquila, are briefly described (Colangelo and De Sortis, 1994). That is an apparatus for 2-d.o.f. specimens whose displacement-control system is based upon electro-mechanic actuators, which have been preferred to hydraulic ones thanks to their lower cost. It is known they also move slower, however this feature often assures an accurate positioning. The pseudodynamic system consists of the following:

- screw-actuator (350kN stationary load; 500kN transient load; 2.2kW electric engine; ±20cm displacement range; 1.45cm per minute speed);
- displacement transducer (0.5μm resolution; ±25cm displacement range; RS232 9600-baud output, 7 measures per second);
- load cell (various types);
- data acquisition electronic unit (60 data channels; 4.44ms A-D conversion time; RS232 19200-baud output);
- multifunction computer board (two analog output channels; sixteen analog input channels; two 8-bit digital I/O parallel ports; 12-bit A-D and D-A converters; 10μs conversion time);
- personal computer and related software.

Pseudodynamic tests are performed in a step-by-step wise: the displacement actuator stops for collecting instrument measures, the next position to be imposed on the specimen is computed by means of an explicit algorithm, then the displacement actuator starts up again.

STEP-BY-STEP INTEGRATION ALGORITHM

The numerical integration of system equations of motion is carried out by resorting to the explicit algorithm proposed by Shing and Mahin (1987*b*). Being planned tests upon specimens of up to 2-d.o.f.s, the use of implicit methods (Shing *et al.*, 1991), which require a somewhat elaborate implementation, is excluded at present. Response quantities at time $(i+1)\Delta t$, with Δt equal to the constant integration time-step, are furnished by the following recurrent expressions:

$$\mathbf{d}_{i+1} = \mathbf{d}_i + \Delta t \mathbf{v}_i + \frac{\Delta t^2}{2}\mathbf{a}_i$$

$$\mathbf{a}_{i+1} = \mathbf{M}^{-1}\left[\mathbf{f}_{i+1} - (1+\alpha)\mathbf{r}_{i+1} + \alpha\mathbf{r}_i\right] + \frac{\rho}{\Delta t^2}\left(\mathbf{d}_i - \mathbf{d}_{i+1}\right)$$

$$\mathbf{v}_{i+1} = \mathbf{v}_i + \frac{\Delta t}{2}\left(\mathbf{a}_i + \mathbf{a}_{i+1}\right)$$

where $\mathbf{d}$, $\mathbf{v}$ and $\mathbf{a}$ in this order are the nodal displacement, velocity and acceleration vectors; $\mathbf{M}$ is the mass matrix; $\mathbf{f}$ is the seismic excitation vector; $\mathbf{r}$ denotes the static reaction of the specimen; α and ρ are numerical parameters. The latter ones improve the Shing-Mahin method with respect to the original Newmark algorithm, since they make it dissipative. In fact, provided $\alpha > 0$ and $\rho \le 0$, according to Bathe and Wilson (1973) the linear solution results oscillatory and stable if it holds:

$$\sqrt{-\frac{\rho}{\alpha}} \le \omega\Delta t < \frac{1+\sqrt{1-\rho(1+\alpha)}}{1+\alpha}$$

where ω stands for circular frequency of the system. The left inequality gives non-negative numerical damping. To assure its fulfillment during pseudodynamic tests, the parameter ρ is made adaptive, following the suggestion by Shing and Mahin (1985).

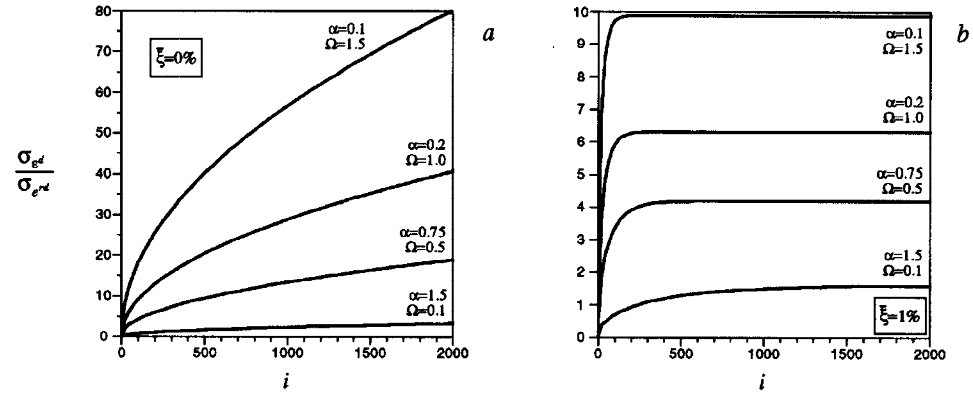

Fig. 1: Experimental error propagation

Concerning experimental error propagation, in (Colangelo and De Sortis, 1994) the formulation by Shing and Mahin (1987) is applied to the present algorithm. The expression of the standard deviation of the error which affects the computed displacement, σ_{ε^d}, to the standard deviation of the measure error of the reaction, $\sigma_{e^{rd}}$, is evaluated at step $i+1$ as:

$$\frac{\sigma_{\varepsilon^d}}{\sigma_{e^{rd}}} = D\left\{\left[\alpha g_{i0}\left(\bar{\xi},\bar{\Omega}\right)\right]^2 + \sum_{j=1}^{i-1}\left[\alpha g_{ij}\left(\bar{\xi},\bar{\Omega}\right) - (1+\alpha) g_{ij-1}\left(\bar{\xi},\bar{\Omega}\right)\right]^2 + \left[(1+\alpha) g_{10}\left(\bar{\xi},\bar{\Omega}\right)\right]^2\right\}^{1/2}$$

where:

$$D = \Omega^2 / \sqrt{B - A^2} \qquad g_{ij}\left(\bar{\xi},\bar{\Omega}\right) = \exp\left[-\bar{\xi}\bar{\Omega}(i-j)\right]\sin\left[\bar{\Omega}(i-j)\right]$$

$$\Omega = \omega\Delta t \qquad \bar{\xi} = -\ln(B)/2\bar{\Omega}$$

$$A = 1 - (1+\alpha)\Omega^2/2 - \rho/2 \qquad \bar{\Omega} = \tan^{-1}\left(\sqrt{B - A^2}/A\right)$$

$$B = 1 - \alpha\Omega^2 - \rho$$

The formula above is valid if computed displacements are used in calculations, rather than the measured ones, and under the assumption of experimental errors as uncorrelated random variables with the same distribution. Fig. 1 shows the aforementioned standard deviations versus the number of integration time-steps, in the case of no dissipation (Fig. 1*a*) and with a small numerical damping $\bar{\xi} = 1\%$ (Fig. 1*b*). It is noted that displacement error results smaller, and stable after a few hundreds of steps, thanks to numerical damping.

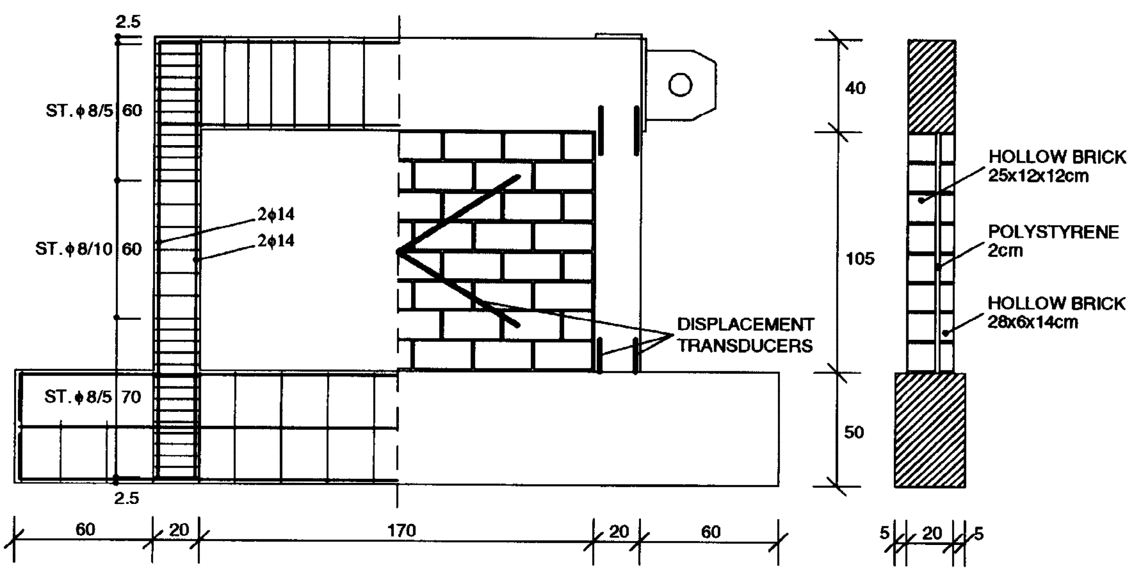

Fig. 2: Specimen

Table 1: Mechanical properties (mean values; units MPa)

Concr.	Reinforc.		Mortar		Brick		Wall				
f_c	f_y	f_t	R_c	f_{ct}	f_{cs}	f_{cw}	f_{cs}	f_{cw}	f_v	E_s	E_w
30.8	553	642	21.4	5.0	23.2	2.5	6.9	3.2	0.41	1740	1380

Notation:

f_c	cylindrical compressive strength
f_y	yielding strength
f_t	failure strength
R_c	prismatic compressive strength
f_{ct}	prismatic flexural strength
f_{cs}	unit/wall compressive strength, strong direction
f_{cw}	unit/wall compressive strength, weak direction
f_v	diagonal compression strength (following ASTM 519E)
E_s	Young's modulus, strong direction
E_w	Young's modulus, weak direction

PILOT EXPERIMENT

The prototype specimen used for pseudodynamic test is sketched in Fig. 2. It is meant to represent the first story of a five-floor residential building, provided with strong beams. Vertical loads are applied to the columns, in the measure of 250kN each one. Reinforcement is designed by assuming a peak of base acceleration equal to 0.25g, response spectrum ordinate equal to 2.5 and behavior (i.e. force reduction) factor equal to 3.

Table 1 lists the most important mechanical properties of constitutive materials and elements. Values in the table refer only to clay brick sized 25×12×12cm (see Fig. 2); that is a vertical-hole brick whose empty percentage results equal to 55.0%. The small walls to be used for qualification tests are sized respecting RILEM Recommendations (1988). The initial stiffness of the infilled specimen is measured equal to 137MN/m, corresponding to a fundamental period of 0.12s.

The pseudodynamic test is carried out with the Tolmezzo earthquake dated 5/6/1976, East-West component, having a peak of acceleration equal to 316cm/s^2 (Fig. 3). The accelerogram is purged of its start and tail as well (see Fig. 3), which are considered of no interest for the prototype experiment. Fig. 4 compares the linear response spectrum of such modified accelerogram with that one proposed by Eurocode 8 (1994) for intermediate soil and damping of 5%, once amplified to the same intensity. Scale problems are touched: since the specimen represents a half-sized model in its plane of an actual structure, the test is performed by contracting the time-scale with the factor $1/\sqrt{2}$.

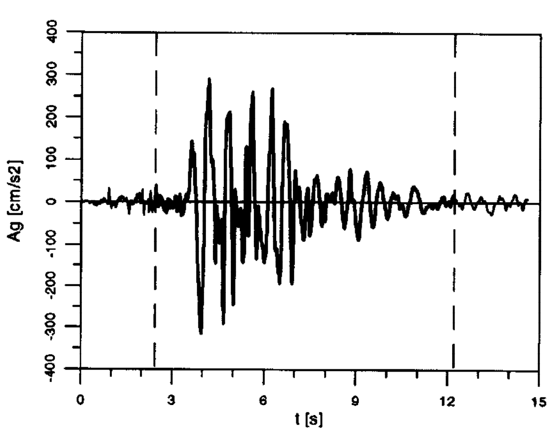

Fig. 3: Tolmezzo earthquake, E-W component

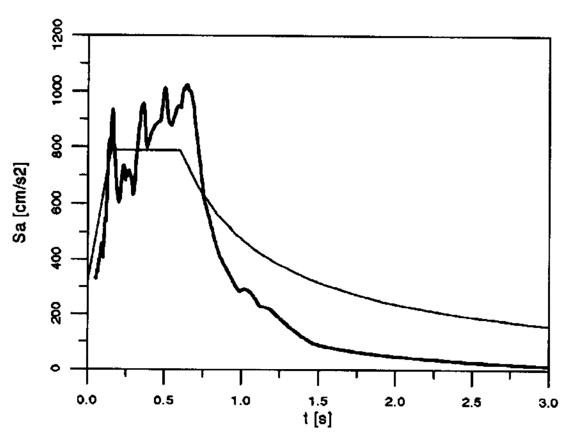

Fig. 4: Response spectra of acceleration

RESULTS

The following Figs. 5–10 illustrate some results of the pilot pseudodynamic test with a thick line. The symbol d denotes the horizontal displacement divided by the clear height of the columns; r is the static reaction of both the wall and r. c. frame on the whole.

Typical shear cracking of the infill (see Fig. 10, where is reported also the numbering adopted for the transducers) become valuable when the two negative peaks of displacement just before and after the time t=2s occur (Fig. 6; see also the accelerogram in Fig. 3). There the measured deformations of the wall reach values of the order of 1‰ (Fig. 9; elongation is positive). Cracking takes place through mortar and bricks as well, due to their similar strength (Table 1). Subsequently, for 2s< t <4s, pinching is visible in the hysteretic force-displacement relationship, together with a certain reduction of the specimen stiffness (Fig. 5*a*). Between instants t=3s and t=4s, reinforcement yielding occurs (Fig. 8), emphasizing cracking in the panel: the displacement transducer 18 measures elongation which will not be recovered (Fig. 9). Then also the strength of the specimen clearly drops down (Fig. 5*b*).

Finally, severe damage is observed in the upper corner opposite to the actuator connection (Fig. 10). There the column displays a full-crossing crack, the exterior of the bricks is collapsed and the infill results visibly moved out of its plane. The residual displacement drift (Fig. 6), conflicting with the last measures of all the transducers except for the 16 and 17 ones, can be explained with a certain observed sliding occurring horizontally along the crack which crosses the top of the column and the beam-infill interface, the column-infill interface on the same side of the actuator having presumably a gap. The residual stiffness of the specimen equals 1/6th of the initial value, giving the ruined infill no contribution.

Figs. 6, 7 e 9 show also simulated results with a thin line. The adopted model (Mondcar and Powell, 1975) consists of linear elastic beam elements provided with plastic hinges located in their terminal sections. The moment-rotation law of the hinges follows the well known Takeda rules (Takeda *et al.*, 1970). The infill is accounted for by introducing two compression-only resistant equivalent struts, whose constitutive relationship is that one proposed by Klingner and Bertero (1978); truss width is assessed by resorting to the expression suggested by Mainstone (1974).

At a first sight, the numerically evaluated response looks far away from the experimental one. However, it should be noted that the curves agree well for the first three seconds of the analysis. In addition, al least qualitatively they appear as very similar until t=4.5s, that is almost for the full duration of the strong phase of the earthquake (see Fig. 3, where the instant in object is t=7s). Conversely, in the remaining range of time decisive differences arise, including clearly distinct fundamental periods, with the experimental one longer.

CONCLUSIONS

It is obvious that a single pseudodynamic test does not justify any conclusion of a general nature. However, on the basis of the work described in this paper, it is believed in the following.

Firstly, the pseudodynamic method proves itself suitable for testing infilled r.c. frames. The statement above is reputed as valid with reference to non-massive infills, where theoretically the lumped-mass scheme remains acceptable, and practically displacement-control problems should not arise due to excessive stiffness of the specimen.

Secondly, the discussion comparing experimental results with the numerical ones suggests the necessity of a more accurate modeling of degrading, if one aims at faithful simulations. Perhaps that is a too ambitious objective operating with such idealizations as equivalent struts, being struts clearly oversimplified models which will never allow to reproduce the influence of some parameters upon the behavior of the infill in reality. However, it should be kept in mind that analyses with the much more refined (and complex in the same proportion) so-called micromodels appear really as unfeasible for actual buildings.

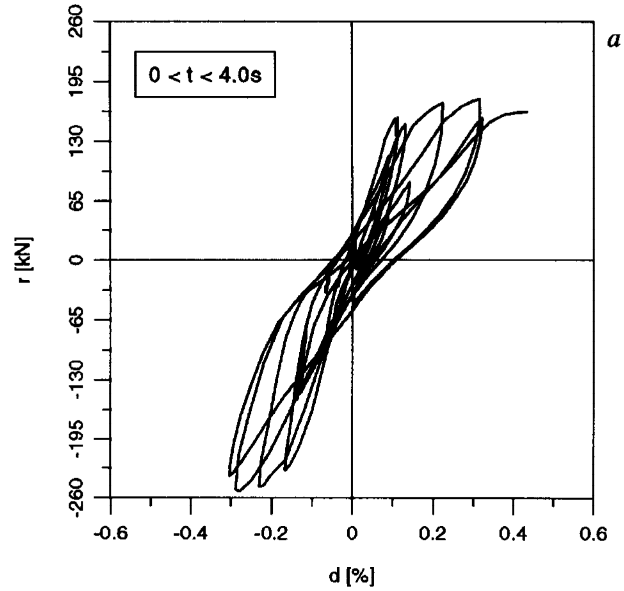

Fig. 5: Force-Displacement relationship

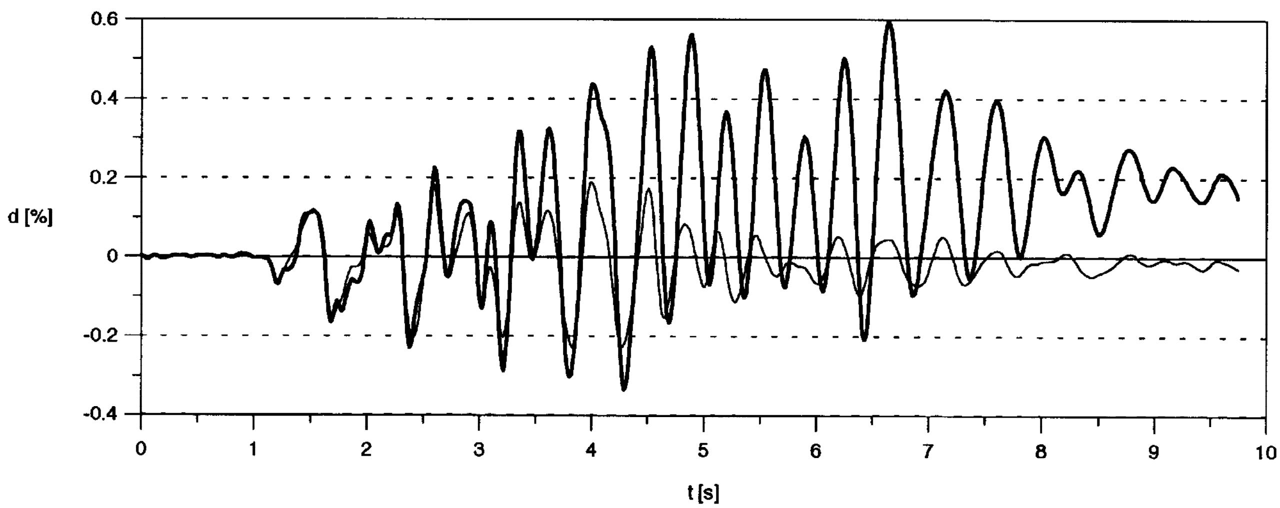

Fig. 6: Displacement time-history

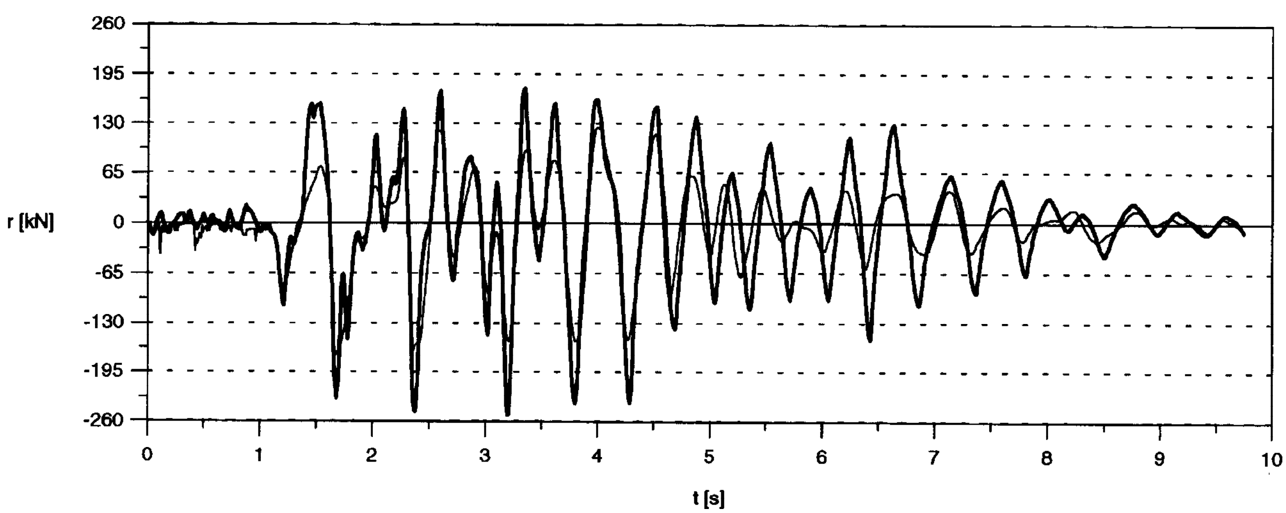

Fig. 7: Reaction time-history

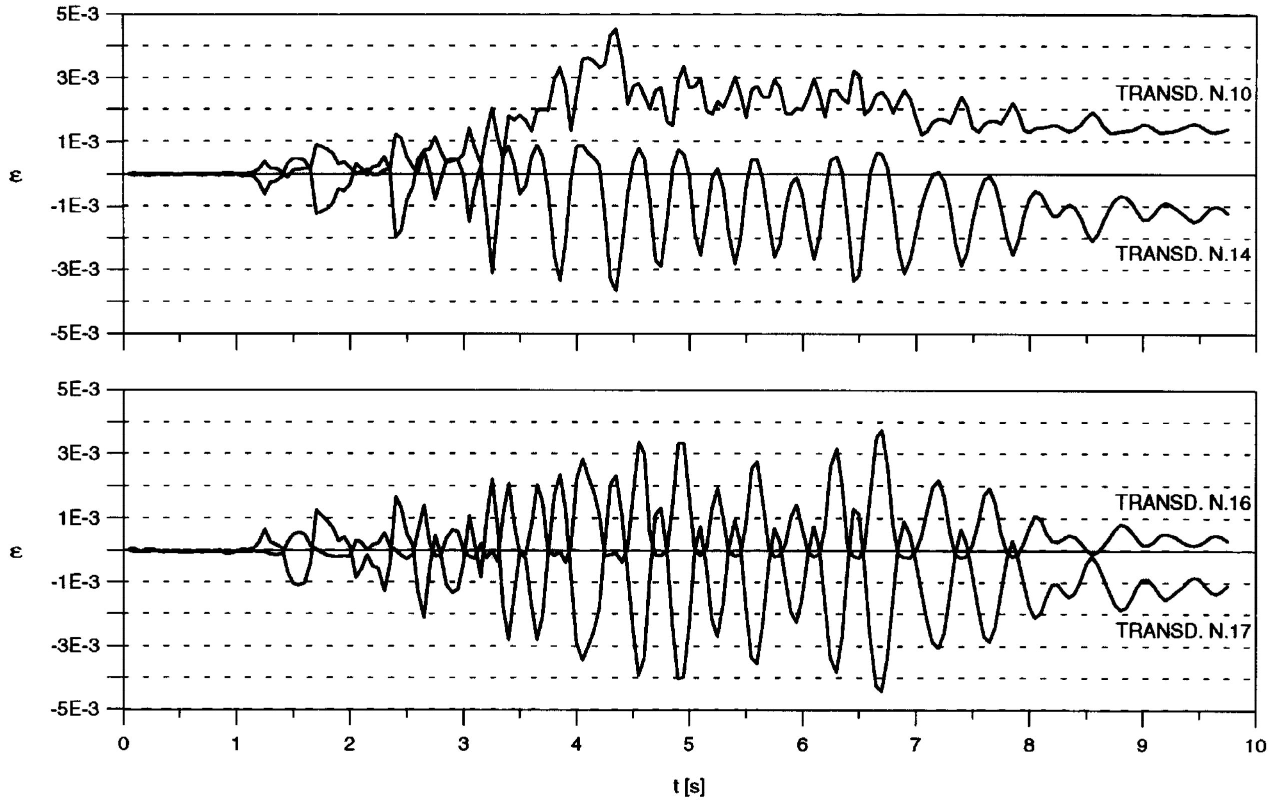

Fig. 8: Reinforcement deformation time-history

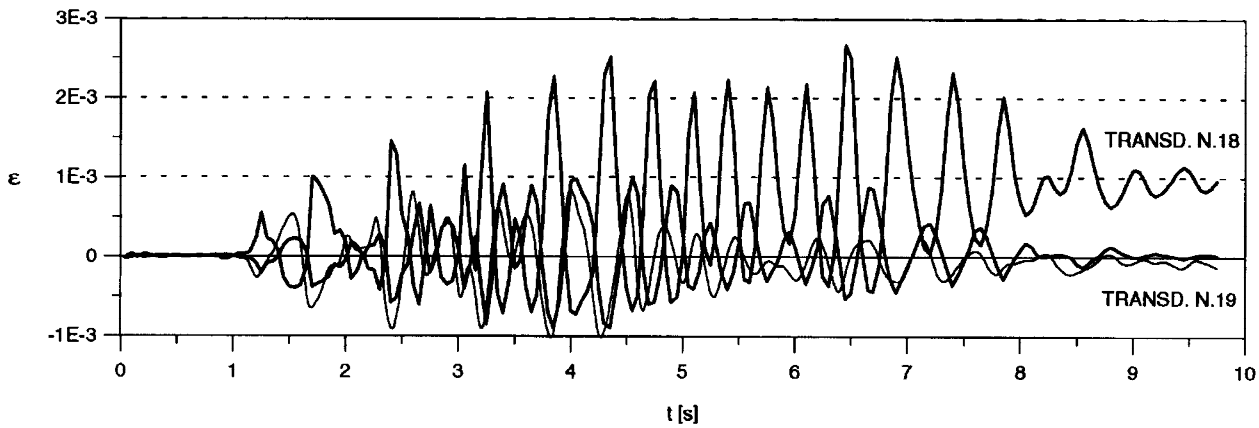

Fig. 9: Infill deformation time-history

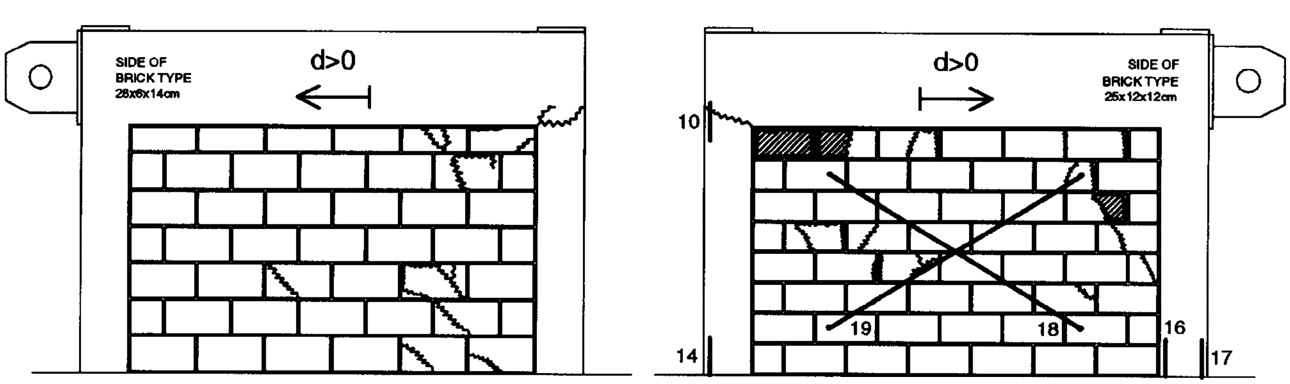

Fig. 10: Cracks and numbering of displacement transducers

Acknowledgment. This work has been partially financed with a grant of the Italian Ministry of the Scientific Research (M.U.R.S.T.).

REFERENCES

Bathe, K. J. and E. L. Wilson (1973). Stability and accuracy analysis of direct integration methods. *Earthquake Engrg. and Struct. Dyn.*, **1**, 283–291.

Brokken, S. T. and V. V. Bertero (1981). Studies on effects of infills in seismic resistant reinforced concrete construction. *Report UCB/EERC-81/12*, Earthquake Engrg. Res. Center, Univ. of California, Berkeley.

Calvi, G. M., G. Via and E. Vintzeleou (1993). Reinforced concrete infilled frames. *C.E.B. Bull. 220*, **2**.

Colangelo, F. and A. De Sortis (1994). Esperimenti pseudodinamici: studio preliminare e realizzazione su sistemi ad un grado di libertà. *Report DISAT 1/94*, Univ. of L'Aquila (in Italian).

Eurocode 8, Part 1-1 (1994). Seismic actions and general requirements of structures. *ENV 1998-1-1*, 18–20.

Eurocode 8, Part 1-3 (1994*b*). Specific rules for various materials and elements. *ENV 1998-1-3*, 46–50.

Klingner, R. E. and V. V. Bertero (1978). Earthquake resistance of infilled frames. *Jnl. of the Struct. Div. ASCE*, **104 (ST6)**, 973–989.

Mahin, S. A. *et al.* (1989). Pseudodynamic test method–Current status and future directions. *Jnl. of Struct. Engrg. ASCE*, **115 (8)**, 2113–2128.

Mainstone R. J. (1974). Supplementary note on the stiffnesses and strengths of infilled frames, *Current Paper CP 13/74*, Building Research Station, Watford.

Mondcar, D. P. and G. H. Powell (1975). ANSR-I: General purpose program for analysis of nonlinear structural response. *Report EERC 75-37*, Earthquake Engrg. Res. Center, Univ. of California, Berkeley.

RILEM Draft Recomm. TC 76-LUM (1988). General recommendations for methods of testing load-bearing unit masonry. *Materials and Struct.*, **21 (123)**.

Shing, P. B. and S. A. Mahin (1985). Computational aspects of a seismic performance test method using on-line computer control. *Earthquake Engrg. and Struct. Dyn.*, **13**, 507–526.

Shing, P. B. and S. A. Mahin (1987). Cumulative experimental errors in pseudodynamic tests. *Earthquake Engrg. and Struct. Dyn.*, **15**, 409–424.

Shing, P. B. and S. A. Mahin (1987*b*). Elimination of spurious higher-mode response in pseudodynamic tests. *Earthquake Engrg. and Struct. Dyn.*, **15**, 425–445.

Shing, P. B., M. T. Vannan and E. Cater (1991). Implicit time integration for pseudodynamic tests. *Earthquake Engrg. and Struct. Dyn.*, **20**, 551–576.

Takeda, T., M. A. Sozen and N. N. Nielsen (1970). Reinforced concrete response to simulated earthquakes. *Jnl. of the Struct. Div. ASCE*, **96 (ST12)**, 2557–2573.