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EFFECT OF INFILLS ON THE GLOBAL SEISMIC BEHAVIOUR OF R/C FRAMES: RESULTS OF PSEUDODYNAMIC AND SHAKING TABLE TESTS

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ABSTRACT

A series of pseudodynamic tests have been conducted on a full-scale four-storey reinforced concrete building designed according to Eurocodes 2 and 8. The building had dimensions of 10 m x 10 m in plan, and was 12.5 m high. A first test was conducted on the bare frame by using an artificially generated earthquake with nominal acceleration 50% larger than the value adopted in design. A second experimental programme was performed to study the influence of masonry infill panels on the global behaviour of the frame. Two pseudodynamic tests have been conducted, with different infill patterns. The first test was performed by infilling the two external frames with hollow brick masonry in all four storeys (uniform infill distribution). The test was then repeated on the structure without infills at the first storey, to create a soft-storey effect. The pseudodynamic tests were complemented by shaking table tests conducted at the University of Bristol to study the importance of out of plane forces in the behaviour of the infill panels.

KEYWORDS

reinforced concrete, infilled frames, regularity, experimental methods, pseudodynamic testing, design codes

INTRODUCTION

A series of pseudodynamic (PSD) tests were conducted on a full-scale four-storey reinforced concrete building designed according to Eurocodes 2 and 8. The building was 10 m long by 10 m wide, and was 12.5 m high. It was designed as a ductility class "High" structure, for typical live loads and for a peak ground acceleration of 0.3 g and medium soil conditions.

A first test was conducted on the bare frame (Negro *et al.*, 1994). The project was carried out within the framework of the European Association of Structural Mechanics Laboratories (EASML), and was designed to assess the adequacy of the damage indicators to be used in the calibration of Eurocode 8. The pseudodynamic test was conducted by using an artificially generated earthquake derived from a real earthquake (1976 Friuli), with nominal acceleration 50% larger than the value adopted in design. The structure performed as expected. The pattern of the measured rotations was that of a weak-beam, strong-column mechanism. The fundamental frequency of the structure after the test was found to be half of the initial value, but the damage was limited and uniformly distributed.

A second experimental programme was conducted as a part of the work of the Prenormative Research in support of Eurocode 8, to study the influence of masonry infill panels on the global seismic behaviour of the frame (Fardis and Calvi, 1994). Two pseudodynamic tests were conducted, with different infill patterns. A test was performed by infilling the two external frames with hollow brick masonry in all four storeys (uniform infill distribution). The test was then repeated on the structure without infills at the first storey, to create a soft-storey effect. The input signal was the same as in the tests on the bare frame.The objective of the tests was to study the effects of the different layout of infills, as well as to calibrate the computer models for the infills to be used in parametric analyses.

The modern seismic codes neglect, or take into account to a very limited extent, the effects of nonstructural masonry panels. Indeed, the masonry panels strongly affect the behaviour of the main structure. In general, the presence of nonstructural masonry panels has a beneficial effect, because they significantly increase the global strength of the structure. On the other hand, they also increase the initial stiffness, so that the inertial forces may be increased to a large extent. The beneficial effect due to the increase of strength may or may not counterbalance the potentially negative effect due to the global stiffening of the structure.

An even more important problem concerning the effects of infills is their distribution. Irregular arrangement in plan and elevation may cause important concentration of damage in the frames, due to torsional effects or to the formation of soft-storey mechanisms.

TEST ON THE BARE FRAME

The general layout of the full-scale reinforced concrete test structure is shown in Fig. 1. The materials used for the specimens were normal-weight concrete C25/30 as specified by Eurocode 2 (EC2, 1984), and B500 Tempcore rebars and welded meshes. The adoption of this kind of steel, which did not fulfil the requirements of the 1988 draft of Eurocode 8 (EC8, 1988) for ductility class High structures in terms of strain at failure (lower than the 12% limit prescribed) and tensile failure strength to yield ratio (lower than the lower bound fixed by EC8, even for ductility class Low structures), was decided because the Tempcore process is gaining the market in Europe. The preliminary design was carried out assuming typical loads (additional dead load 2.0 KN/m^2, to represent floor finishing and partitions, and live load 2.0 KN/m^2), and high seismicity (peak ground acceleration 0.3g, soil type B, importance factor=1).

The structure satisfied the regularity requirements both for vertical and horizontal configuration, so it was designed as a frame of high regularity in ductility class High, with the behaviour factor q=5. An artificial accelerogram, derived from the 1976 Friuli earthquake, was adopted. The reference signal and the corresponding 5% damping elastic response spectrum are given in Fig. 2. The test was performed using the reference signal multiplied by an intensity factor of 1.5. The nominal peak ground acceleration was then 1.5x0.3=0.45 g, a value which was thought to be representative of the maximum seismic actions for which

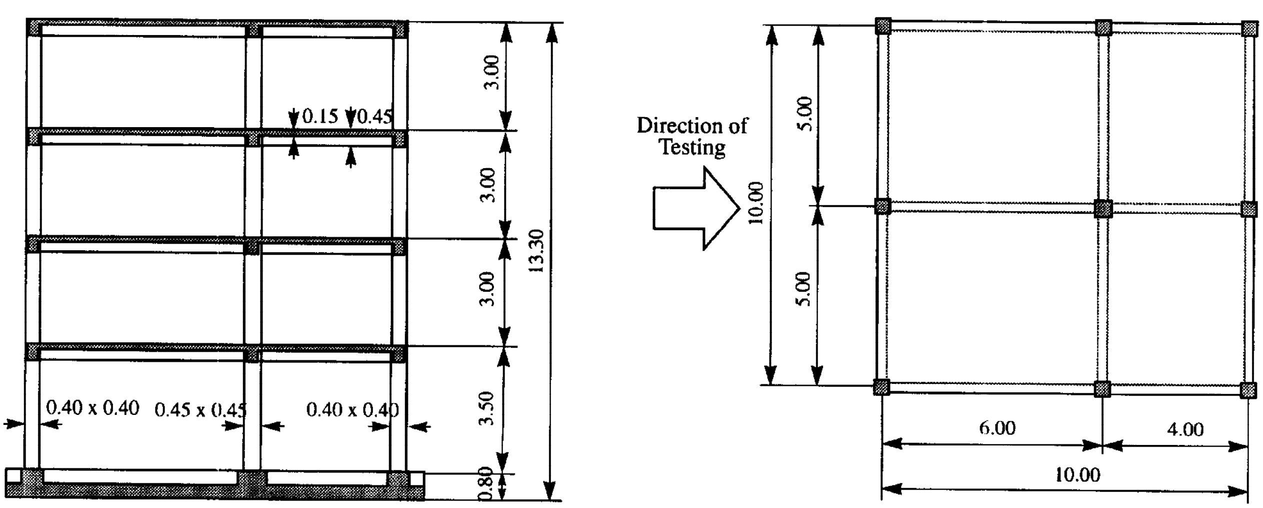

Fig. 1 - Layout of the specimen (dimensions in metres).

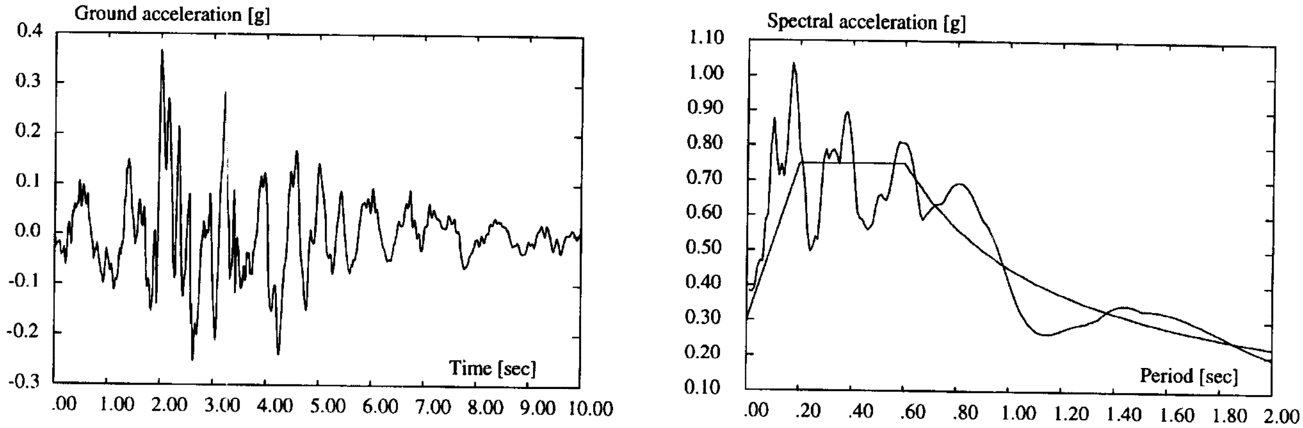

Fig. 2 - Reference accelerogram and elastic response spectrum compared with EC8.

the frame had been designed.

The structure performed very well, the only concern being the apparently low damage sustained. During the test, cracks opened (and closed) in the critical regions of the beams of the first three storeys and of most of the columns. Only the cracks at the beam-to-column interfaces remained permanently open. Neither spalling of the cover, nor local instabilities of reinforcement were observed. Beside the cracks at the beam-to-column interface, which were apparent in the first three storeys and represented evidence of local yielding of the rebars, the specimen remained apparently undamaged.

A direct stiffness measurement was performed before and after the pseudodynamic test. With the stiffness matrices, the vibration eigenfrequencies were computed. The resulting fundamental frequency (0.82 Hz) was more than twice as small as that of the virgin structure (1.78 Hz), indicating a change in stiffness far beyond the progression of cracking. The resulting mode shapes, however, were close to those of the virgin structure. The fact that the mode shapes did not significantly change, even though the changes in the stiffness matrix of the specimen were significant, provides evidence that the structure was uniformly damaged.

The analysis of the maximum rotations measured at the potentially critical locations confirmed the uniform damage pattern. Other local measurements highlighted the important role played by the slippage of rebars in the internal joints. This effect was probably made more severe by the adoption of the Tempcore steel. It resulted in significantly pinched force-displacement loops, and was found to be responsible for most of the differences between experimental results and preliminary numerical predictions.

TEST ON THE UNIFORMLY INFILLED FRAME

Due to the relatively low damage suffered by the structure as a result of the bare-frame tests, no repair actions were taken. The materials to be used for the construction of the infill panels were selected as representative of typical light nonstructural masonry. To facilitate the construction, blocks commonly available in Italy were adopted. The blocks had dimensions of 245 x 112 x 190 (h) mm, with vertical holes taking 42% of the gross section. The average compressive strength of the blocks was found to be respectively 13.3 MPa in the direction parallel to the holes (vertical), and 3.3 MPa in the direction orthogonal to the holes. The mortar was also selected as typical, to reach a compressive strength of 5 MPa (cement, lime and sand in the proportion 1:1:5).

Tests on small specimens were performed at the University of Pavia to measure the resulting properties in the strong and weak directions, as well as in the diagonal direction. For the strong (vertical) direction, a mean strength of 7.3 MPa and a mean modulus of elasticity of 8210 MPa were obtained. The compressive tests in the direction orthogonal to the holes yielded a mean strength of 2.4 MPa and a modulus of elasticity of 2515 MPa. A mean value of the tensile strength of 0.28 MPa and a shear modulus of 1240 MPa were

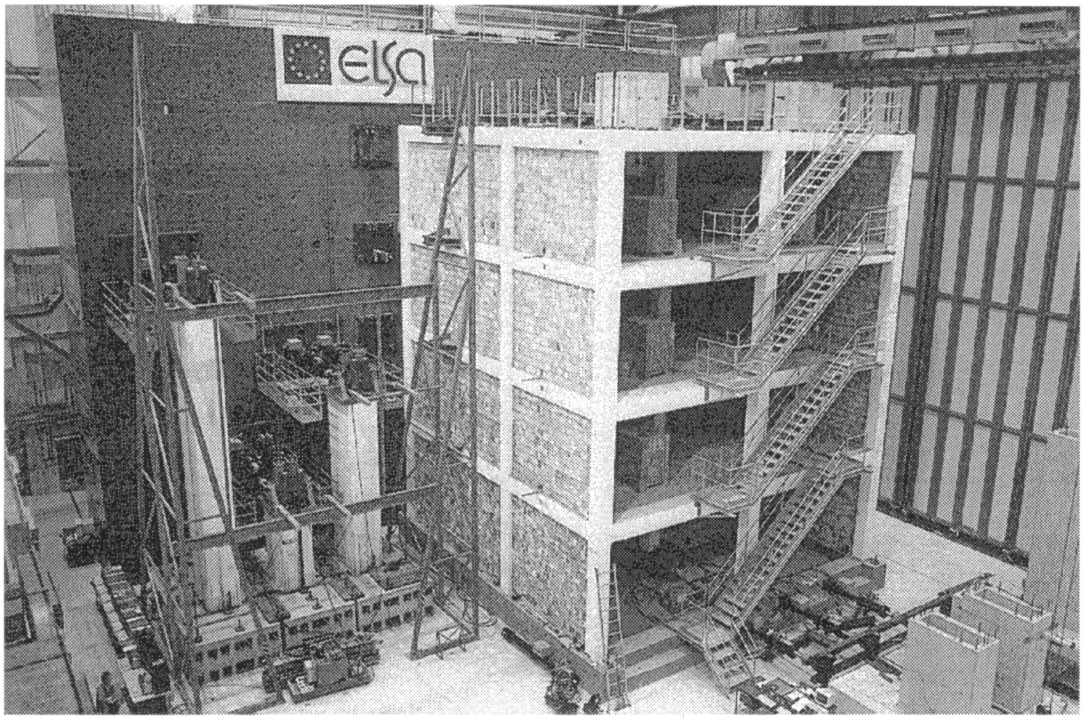

Fig. 3 - R/C frame with uniform infill distribution.

derived from the results of the diagonal compression tests.

Infill panels were placed at all storeys of the external frames (Fig. 3). No special provisions were made in the construction of the panels, so that complete bond between the panels and the frame can be assumed at the beginning of the test. No plaster was placed on the surface of the panels. Before the test, a direct measurement of the stiffness matrix was carried out by displacing in turn each of the storeys by a small quantity (±0.16 mm), while holding the others still. From the stiffness matrix, the corresponding elastic vibration frequencies were computed. The fundamental frequency was found to be 3.34 Hz (for the bare frame this was 1.78 Hz before the test, and 0.82 Hz after the high-level test).

The damage in the masonry panels was a decreasing function of the height: at the first storey, the panels were completely destroyed; at the second storey, they suffered extensive damage (cracking and crushing at the corners); at the third storey, some cracks occurred but no crushing was observed; the panels of the upper storey remained essentially intact. Redundant measurements were taken at each panel up to failure to assist in the calibration of the global computer models to be used in the interpretation calculations.

TEST ON THE SOFT-STOREY FRAME

The masonry panels were demolished and replaced with new ones, leaving the first storey naked. This was expected to lead to a soft-storey mechanism, and corresponds to the case of "drastic reduction of infills in one or more storeys" in Eurocode 8. For such cases, the code requires a local increase of the forces to be used in design, as well as an increase in the portion of the columns of the ground floor to be detailed as for critical regions, up to the entire length of the column. However, none of these requirements was considered in design.

The fundamental frequency from the stiffness measurements turned out to be 1.67 Hz, relatively close to the initial frequency of the bare frame. The high-level test was repeated as for the uniformly infilled frame.

As expected, the test resulted in a concentration of drift at the ground floor, and some damage was suffered by the panels of the second storey only. As for the previous test, local measurements were taken at some of the masonry panels. In addition, the rotations at the ends of the columns and beams of the ground and first floors were monitored.

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DISCUSSION OF THE RESULTS

The comparison of the results obtained in the tests with different infill configurations is particularly meaningful, because most of the design codes neglect the changes due to the presence of the infills.

The measured storey displacements are shown in Fig. 4. The maximum top displacement obtained for the bare frame (about 210 mm) proved to be comparable with the one obtained for the soft-storey structure, even though the spatial distribution of the storey drifts was obviously quite different. In spite of the almost complete failure of the infill panels, the maximum top displacement experienced by the uniformly-infilled frame was more than 2.5 times smaller.

The time histories of the base shear obtained for the three tests are also depicted in Figure 4. The maximum base shear for the soft-storey structure is only slightly larger than that of the bare frame (1.45 MN). The maximum base shear obtained for the uniformly infilled frame was almost 50% larger than that of the bare frame. This result is particularly interesting, since in most cases the presence of light infills is not expected to significantly increase the global strength of the structure.

The storey-level hysteretic loops (Fig. 5) provide some insight into the behaviour of the structure. The storey shear vs inter-storey drift diagrams for the bare frame exhibit stable dissipative loops, with amplitude progressively decreasing from the first to the top level. Even though the loops take a pinched shape after the first large-amplitude cycle - the main reason for this effect having been identified in the loss of bonding in the internal joints - no strength deterioration occurs. For the case of the uniformly-infilled frame the amplitude of the cycles also decreases with the storey level, however, severe strength and stiffness deterioration - typical of the infill behaviour - can be noticed. For the case of the soft-storey frame, the energy dissipation is almost limited to the ground floor, and the onset of pinching in the shape of the loops after the first amplitude cycle is as evident as in the loops of the bare frame.

As far as the inter-storey drifts are concerned, the values for the uniformly-infilled frame are less than half of those of the bare frame. For the soft-storey structure, most of the interstorey drift took place at the bottom storey, where a value larger than 3.5% was achieved. The analysis of the rotations measured at the end of beams and columns highlighted a uniform strong-column weak-beam mechanism for the bare frame, and a bottom-storey sidesway mechanism for the soft-storey infilled frame, with large rotations at the column ends (33 mRad). Apparently, the uniformly infilled frame did not suffer significant nonlinear deformation, even though post-test numerical analyses have shown that local sidesway mechanisms may have been activated.

SHAKING TABLE TESTS

The University of Bristol conducted a series of shaking table tests to investigate the strength of simple, unreinforced masonry panels when subjected to combined in-plane and out-of-plane forces. The main scope of the tests was to identify the effects of the out-of-plane forces on the in-plane behaviour of the panels, and to verify the possibility of premature failure of the panels due to the out-of-plane forces. Only non-structural masonry infill panels were of interest. Four panels were shaken with a variety of input motions applied in-plane, out-of-plane and combination, having in mind to detect the onset of failure and the associated acceleration levels for the different inputs.

The panels, measuring 1.5m high by 2.0m wide, had a single wythe 100mm thick, giving a thickness ratio of 16:1. Both standard (compressive strength 25 MPa) and lightweight (density 700 kg/m^3, compressive strength 2.3 MPa) blocks were used, with mortar having nominal compressive strength ranging from 1.0 to 16.0 MPa. The panels were built following normal site practice inside a stiffened steel portal frame with semi-rigid joints. The bottom course of bricks was laid onto the roughened concrete base, and the vertical joints between the painted column and the masonry were filled with mortar, without ties or other bonding enhancements. Kentledge of 6t was placed on top of the portal frame to provide in-plane inertial forces. The kentledge was supported by an independent structure, and the masonry walls were constructed after placing the kentledge, to prevent the kentledge load from being applied to the wall. The setup was designed to remove any out-of-plane forces due to the lateral sway of the frame. Out-of-plane sway and toppling of the kentledge were constrained by steel braces, so that out-of-plane forces only arised from rigid body transla-

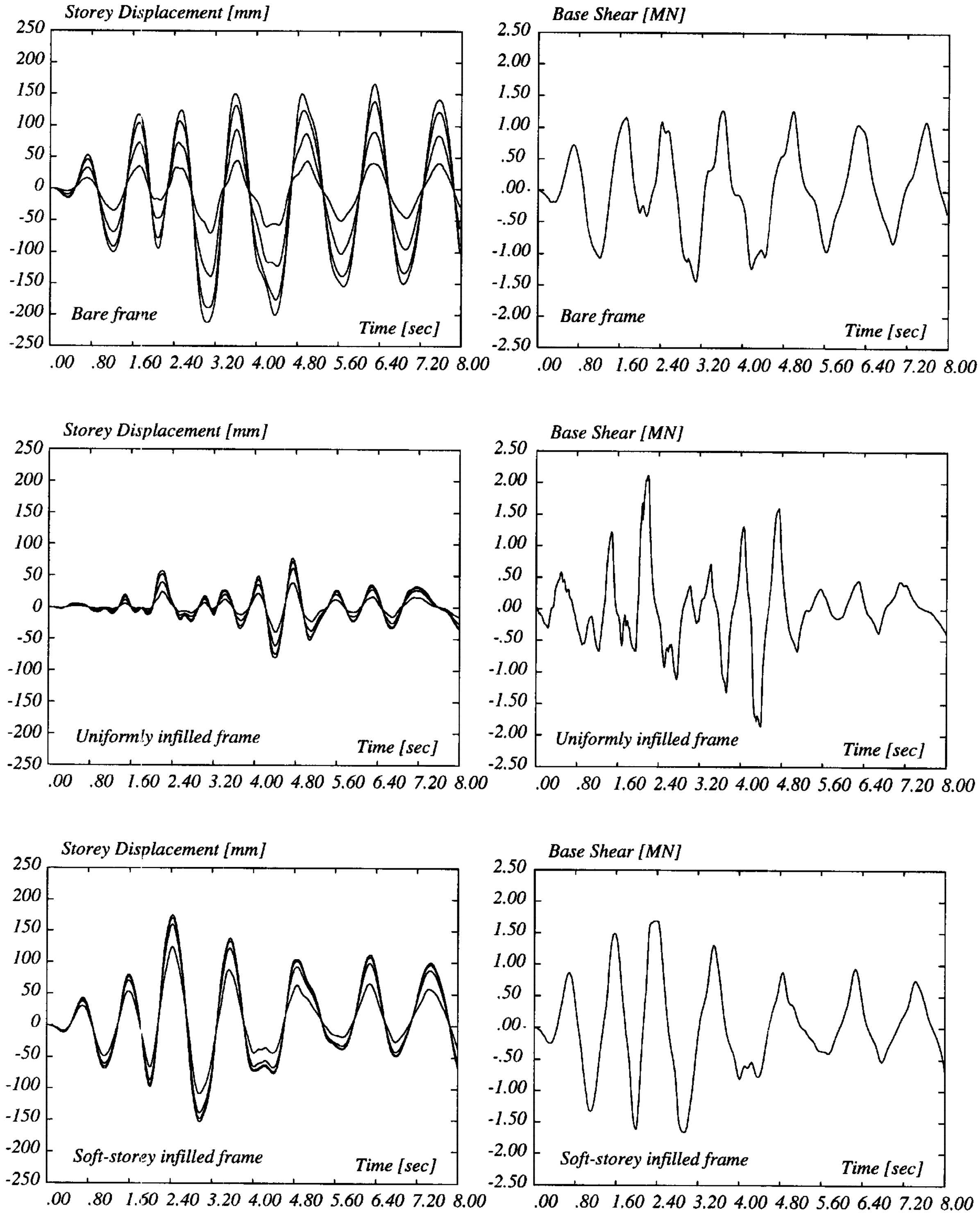

Fig. 4 - Measured storey displacements and base shears for bare, uniformly infilled and soft storey infilled frames.

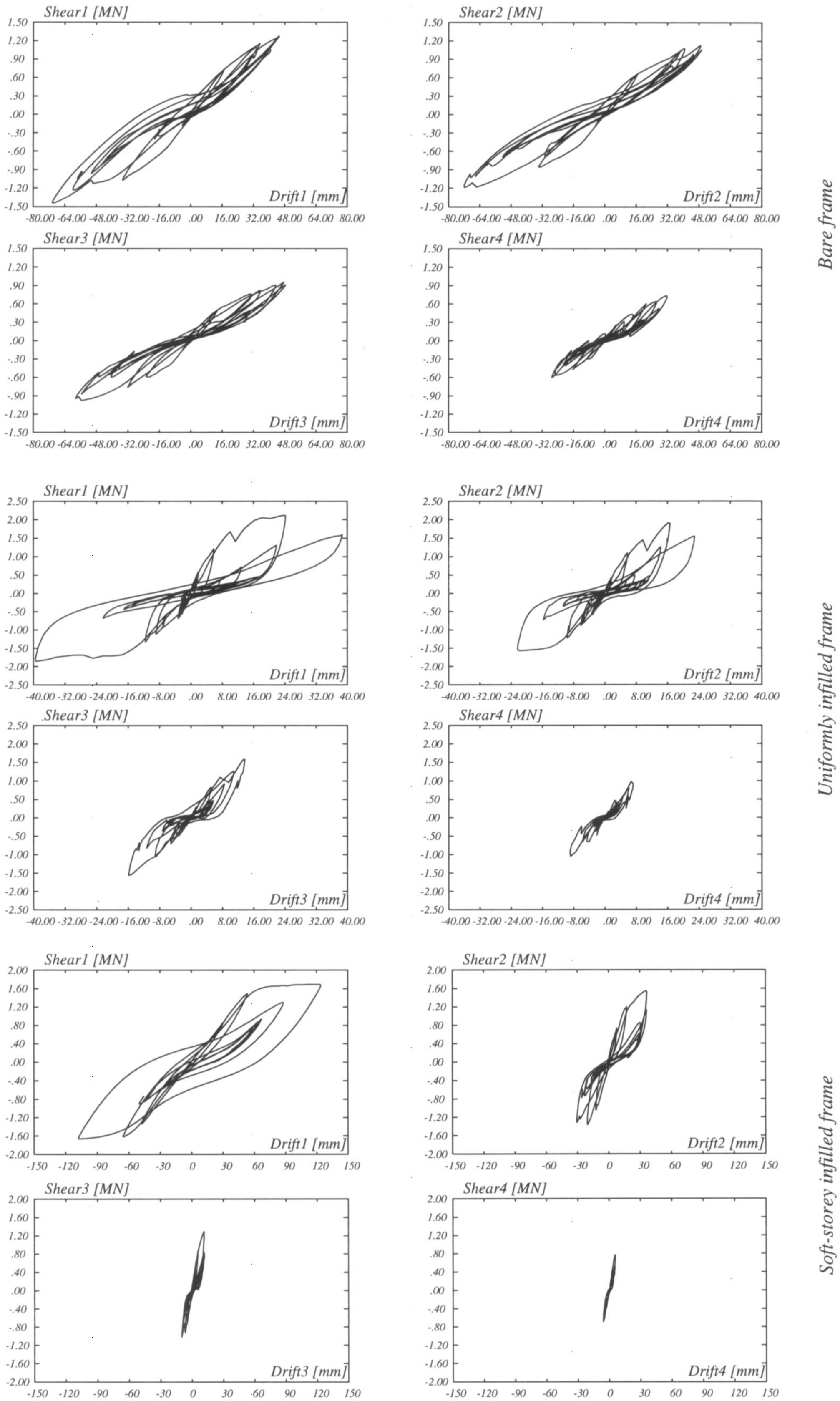

Fig. 5 - Storey-level hysteretic loops.

tion of the whole apparatus in this direction. Details of the test apparatus are given in (Taylor and Ndamage, 1995).

Simple sine dwell motions were used in the first three tests. These comprised a constant sinewave ramping up to maximum amplitude over ten cycles, holding this amplitude for twenty cycles before ramping down to zero over ten cycles. The final series of tests used table motions fitting the EC8 response spectra. Different sequences of tests were conducted with increasing amplitude.

The first panel was constructed from lightweight blockwork and a very weak mortar. It was shaken initially in the in-plane direction, followed by out-of-plane and combined shaking. Although the panel was horizontally cracked in-plane, with the two parts of the panel sliding over each other along the failed course, and although a gap formed between the panel and column, it did not collapse, even when shaken by out-of-plane motions in excess of 2g. This was due in part to the low density of the panel, and in part to the confinement at the top joint.

The second panel was built from lightweight blockwork and mortar of about the same strength of the blocks. It failed in-plane by diagonal cracking. Once again, it did not collapse despite experiencing accelerations in excess of 1g in over 15 earthquakes.

A third panel was built from standard concrete bricks using a weak mortar. The first crack occurred horizontally at the base of the panel. When the sine dwell amplitude of the in-plane motion was increased, further diagonal cracking along mortar joints occurred. The panel was subjected to a total of 26 shakes, including in-plane, out-of-plane and combined sine dwells and seismic inputs. Peak accelerations reached 2.9g in both axes. Even though cracks developed around almost every brick, and out-of-plane accelerations exceeding 10g were measured at the centre of the panel, the confinement was still sufficient to prevent the collapse.

CONCLUSIONS

The full-scale pseudodynamic tests confirmed that the presence of light nonstructural masonry infills can change the response of the structure to a large extent. The presence of a regular pattern of infills considerably prevents energy dissipation from taking place in the frame. However, the progressive complete failure of the panels may lead to the activation of partial sidesway mechanisms. Irregularities in the panels result in unacceptably larger damage in the frame. In general, the effect of the nonstructural infill panels cannot be neglected in design.

The shaking table tests suggested that the out-of-plane collapse of nonstructural infill panels can be controlled by good detailing and site practice. This is true even for weak mortar joints, provided the joints around the panel are thin (about 10 mm).

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