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FORCED VIBRATION TESTS ON A FOUR-STOREY REINFORCED CONCRETE FRAME BEFORE AND AFTER PSEUDO-DYNAMIC LOADING

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ABSTRACT

Masonry infill panels can have significant positive and negative effects on the overall seismic behaviour of frame structures, although these effects are largely ignored in seismic codes including Eurocode 8. This paper describes a series of four steady-state forced vibration tests conducted on a full-scale four-storey reinforced concrete frame with plan dimensions of 10m by 10m and an overall height of 13.3m. The tests were conducted before and after pseudo-dynamic tests and both with and without masonry infill panels. These tests showed that masonry infill panels have a very significant effect on the overall seismic behaviour of frame structures. In the direction in which the panels were added, the frequency of the fundamental mode increased from 0.96 Hz to 3.8 Hz, and there was an almost fivefold increase in the modal mass, producing a modal stiffness about 70 times higher. There were similar increases in the natural frequency, modal inertia and modal stiffness in the fundamental torsional mode.

KEYWORDS

Forced vibration test; masonry infill panels; concrete frame;

INTRODUCTION

Modern seismic codes, including Eurocode 8, take very little account of the effects that non-structural masonry panels have on the overall seismic behaviour of reinforced concrete structures. These panels have a generally positive effect in that they increase the resistance to lateral loads. On the other hand they also increase the initial stiffness of the structure, so increasing the inertial forces. Antisymmetric distribution of infills can also lead to larger torsional response or soft-storey effects. The work described in this paper is part of the prenormative research in support of Eurocode 8 to assess these effects realistically so that future versions of the code can incorporate them.

DESCRIPTION OF THE FRAME

The test structure consists of a four-storey reinforced concrete frame which was designed to Eurocodes 2 and 8 and built at the European Commission's Joint Research Centre at Ispra in Italy.

The frame is 10m by 10m in plan and 13.3m high overall (Fig.1). The ground storey is 3.5m high, the other storeys are 3.0m high, and the base is 800mm thick. The structure is symmetric in the north-south direction with two 5.0m bays, but has bays of 6.0m and 4.0m in the east-west direction. The exterior columns are 400mm x 400mm, the single interior one is 450mm x 450mm. The beams are all 450mm high and 300mm wide. The solid slab floor used on all four storeys is 150mm thick. Further details of the frame are given by Donea et al, 1995. The reaction wall which was used to carry out pseudo-dynamic tests lies to the west of the frame. Two levels of pseudo-dynamic test were applied to the structure: low-level (0.4 times) and high-level (1.5 times) the reference earthquake signal, which had a ground acceleration of just under 0.4g and a spectrum which corresponded well with the Eurocode 8 spectrum.

TESTING SCHEDULE

Steady-state forced vibration tests were carried out on the frame on four occasions to determine its dynamic characteristics. Each test took less than three working days and was conducted after the access stairs and the hydraulic actuators were removed so that the frame was able to behave in an unrestrained way. The first two sets of tests (A and B) were carried out on the bare frame. The third and fourth sets of tests (C and D) were carried out on the frame complete with masonry infill panels on the exterior walls in the east-west direction only, that is along the same axis as the pseudo-dynamic load was applied (Fig. 2). The first set of tests (set A) was undertaken after the frame had been subjected to a low-level pseudo-dynamic test. Test set B was carried out after the frame had been damaged to some extent after a high-level pseudo-dynamic test. Although there was some visible damage to the frame, this was not of sufficient magnitude for the structure to be repaired. Test set C was performed after the infill panels had been built but before any other tests were conducted on the frame. The fourth set of tests (set D) was carried out after the frame had been subjected to three further test stages. These were a high-level pseudo-dynamic test which caused considerable damage to some of the infill panels, an out-of-plane loading test after which the panels at the ground floor were removed completely and the damaged infill panels which remained were rebuilt, and a further low-level pseudo-dynamic test. Unfortunately a forced vibration test at one of the intermediate stages between tests C and D could not be accommodated within the overall test programme. This makes interpretation of the differences between tests C and D very difficult.

Fig. 1. The bare frame

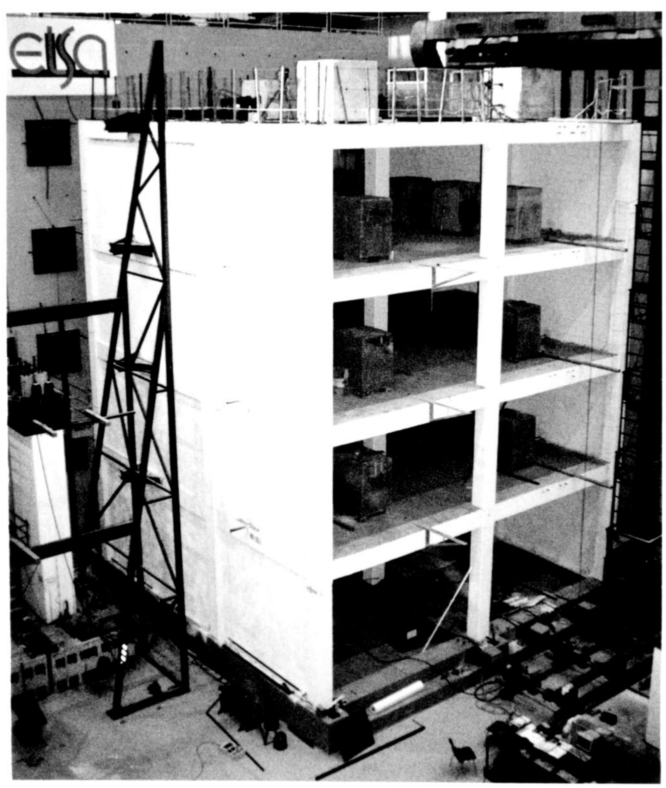


Fig. 2. The frame with infill panels

FORCED VIBRATION TESTING

The Building Research Establishment (BRE) vibrator system can be operated from 0.3 to 20 Hz in steps of 0.001 Hz, and the frequency of the eccentric-mass exciters maintained to an accuracy of better than 0.0005 Hz. Different weights, which range from 3.7 kg per set to 145 kg per set, can be attached to the exciters. The 145 kg set has five unit weights on each of the four arms of the exciter. As well as these basic unit weights there are half, quarter and one eighth unit weights so that a large number of different forces are possible. With the maximum number of weights attached (5 sets), the system can be operated up to 3.16 Hz, and is capable of producing a force of up to 8.4 tonnes peak to peak (at 3.16 Hz). Full details of the exciters and the general testing procedure used by BRE are given elsewhere (Littler 1993, 1989). Two exciters were attached to the top floor of the structure in each of the tests on the concrete frame at Ispra (Fig. 3).

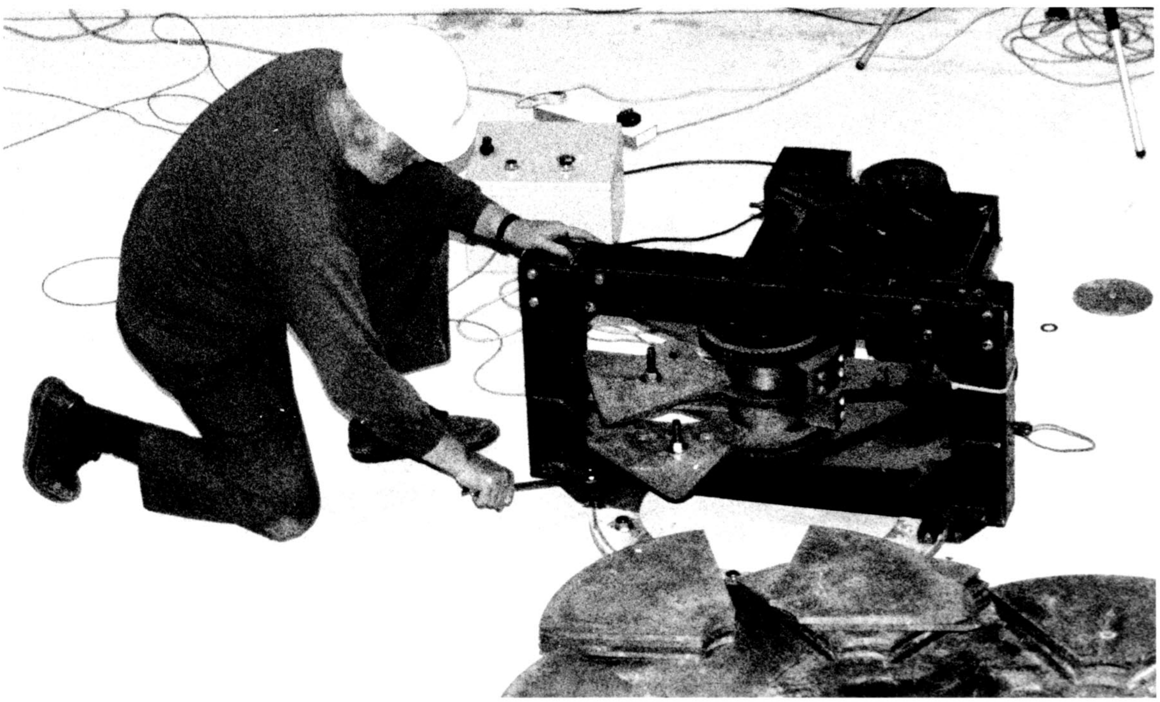


Fig. 3. One of the eccentric-mass exciters

Figure 4 shows an example of one of the response spectra obtained from the first set of tests on the frame. This is the spectrum obtained for the fundamental north-south mode. The best fit theoretical single-degree-of-freedom curve to the experimental data is also shown. The natural frequency and damping values for the theoretical curve are 1.55 Hz and 1.01% critical respectively. The fit between the experimental data and the theoretical curve is not perfect, the former has a negative skew compared with the latter. However, this result is typical of those obtained in these tests and shows one aspect of the non-linear behaviour.

Figure 5 shows an example of the method of obtaining a damping value from a decay of oscillation. This is for the same mode as shown in Fig. 4. The upper part of Fig. 5. shows the decay of oscillation obtained when the vibrator, having been running at 1.542 Hz (the frequency of peak response shown in Fig. 4) was switched off. This decay was measured by an accelerometer aligned north-south in the centre of the top of the frame, where the response while the vibrator was running was 1.93mm peak to peak. The lower part of Fig. 5. shows the best fit theoretical decay to part of the experimental data starting from 3.12 seconds after the beginning of the data shown in the upper part of Fig. 5. This best fit theoretical decay had a damping value of 1.73%, and is an excellent match with the experimental decay.

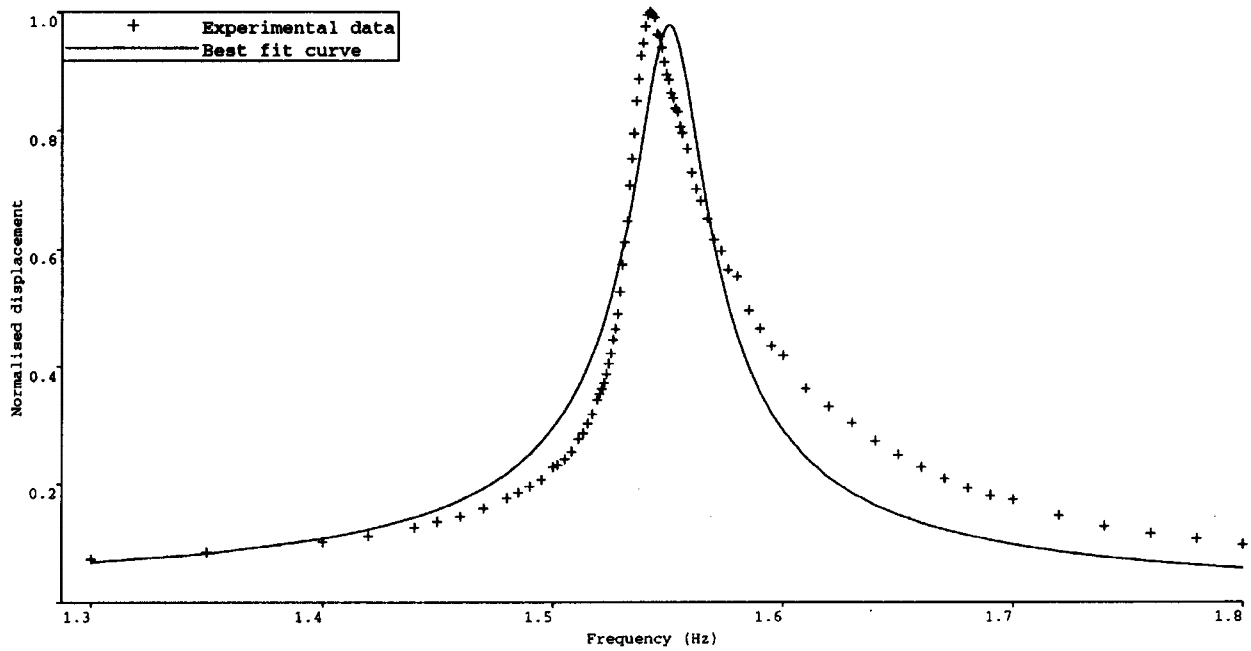


Fig. 4. Curve fitting around the fundamental north-south mode as identified in the first set of steady-state forced vibration tests

Full plan and elevation mode shapes were obtained for all the modes. As an example, Fig. 6 shows how the plan and elevation mode shape for the fundamental east-west mode changed over the sets of tests. This clearly shows the soft-storey effect in test set D.

Table 1 shows how the frequency of the fundamental east-west mode (EW1) fell between tests A and B, but then rose dramatically by test C when the infill panels were added, before falling again in test D to something close to the values found in test B. The figures for the fundamental torsional mode ($\theta 1$) give a broadly similar pattern, but the figures for the fundamental north-south mode (NS1) show little change between tests. The figures for the second order modes show a similar pattern to those for the fundamental modes. The frequency of the second order east-west mode fell from 4.26 Hz in test A to 3.10 Hz in test B. The equivalent frequencies for the second order torsional mode were 5.49 Hz and 4.12 Hz. However, neither of these modes could be identified in test set C. This was because the frame had stiffened considerably after the addition of the infill panels and the frequency of these modes had risen beyond the safe working frequency of the exciters. The frequencies of these modes in test set D were 3.98 Hz and 4.83 Hz. The frequency of the second order north-south mode in the four tests was 4.90 Hz, 3.67 Hz, 4.11 Hz and 3.55 Hz.

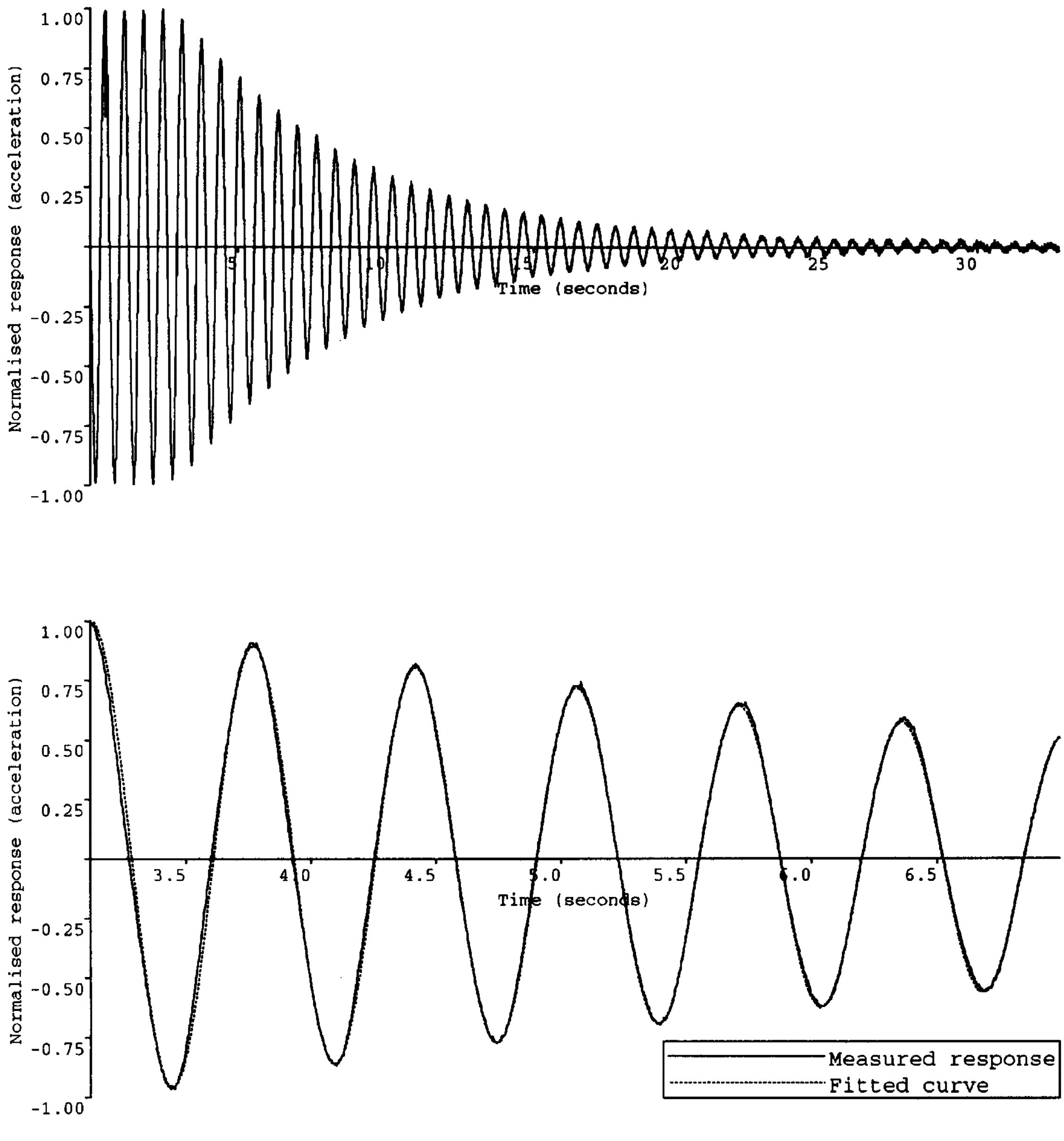
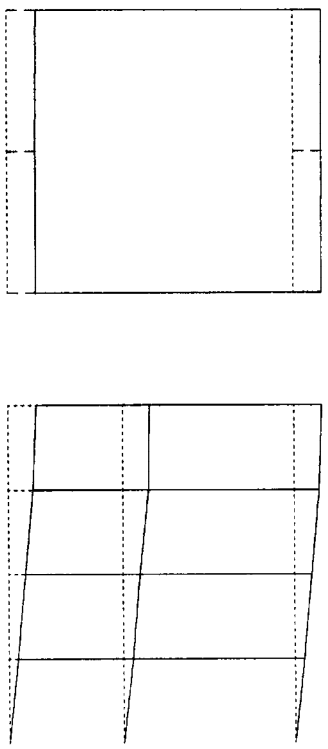
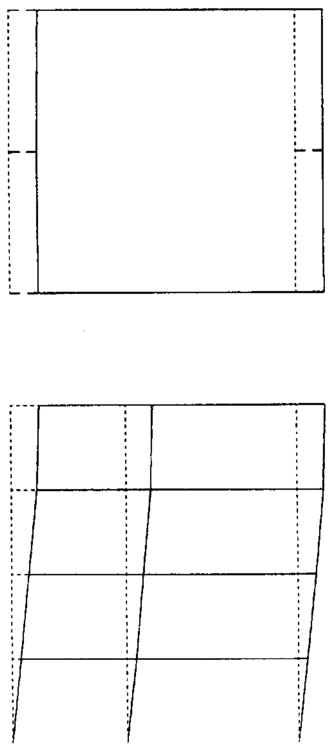


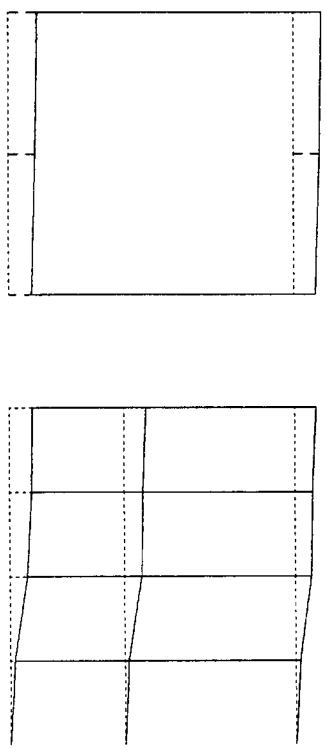
Figure 5 Decay of oscillation in the fundamental north-south mode with best fit theoretical decay to part of it



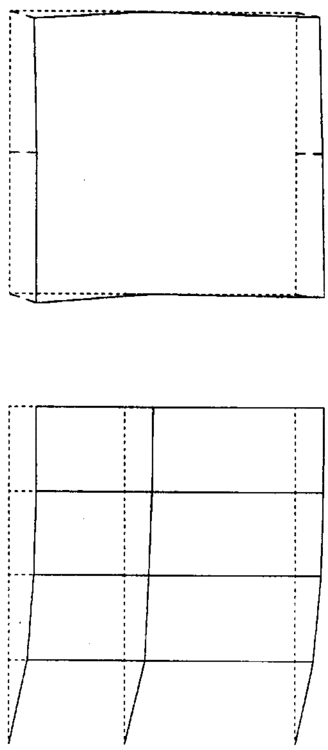
Test A: 1.41 Hz



Test B: 0.99 Hz



Test C: 3.88 Hz



Test D: 1.03 Hz

Figure 6 Normalised plan and elevation mode shapes
for the fundamental east-west mode

Table 1. Dynamic characteristics of the fundamental modes

Test Set	Wts	Highest normalised response (peak to peak)					Decay	Curve fitting		Modal Mass	Modal Stiff
		Freq (Hz)	Force (N)	Acc (m/s^2)	Disp (mm)	Disp/ Force (m/N) (E-6)	Damp (%crit)	Freq (Hz)	Damp. (%crit)	Kg (E+3)	MN/m (E+6)
EW1											
A	1/2	1.414	827	.0904	1.15	1.38	3.01	1.427	1.79	152	12.0
B	1	0.990	811	.0602	1.56	1.92	3.95	1.005	2.63	171	6.60
B	2	0.927	1420	.110	3.25	2.29	4.05	0.937	2.41	159	5.41
B	1/2	1.004	417	.0330	0.828	1.99	3.47	1.013	1.96	182	7.25
C	1	3.880	12500	.209	0.352	.0282	2.25	3.911	3.43	1330	790
C	1/2	3.760	5850	.111	0.199	.0340	3.19	3.674	5.65	826	461
D	1	1.096	994	.0565	1.19	1.20	3.61	1.112	2.58	244	11.6
D	2	1.034	1770	.0906	2.15	1.21	3.70	1.047	2.47	264	11.1
NS1											
A	1/2	1.542	983	.181	1.93	1.96	1.74	1.551	1.01	156	14.6
B	1	1.229	1250	.153	2.57	2.06	2.51	1.240	1.72	163	9.70
B	1/2	1.249	645	.0860	1.40	2.16	2.26	1.261	1.68	166	10.2
B	2	1.185	2320	.265	4.78	2.06	2.88	1.193	1.82	152	8.43
C	1	1.342	1490	.179	2.51	1.69	2.24	1.356	1.55	186	13.2
C	1/2	1.334	736	.106	1.51	2.05	1.91	1.343	1.20	182	12.8
D	1	1.050	912	.118	2.70	2.95	2.07	1.056	1.05	187	8.13
Test Set	Wts	Highest normalised response (peak to peak)					Decay	Curve fitting		Modal Inertia	Modal Stiff
		Freq (Hz)	Torque (Nm) (E+3)	Acc (rad/s^2)	Disp (rads) (E-3)	Disp/ Torque rad/Nm (E-6)	Damp (%crit)	Freq (Hz)	Damp (%crit)	(Kgm^2/ rad) (E+3)	(MNm/ rad)
θ1											
A	1/2	1.826	4.83	2.04	15.5	3.21	2.55	1.838	1.26	46.4	6.11
B	1	1.368	5.43	1.60	21.7	4.00	3.36	1.383	1.88	50.5	3.73
B	2	1.289	9.63	2.79	42.5	4.42	3.26	1.298	1.68	52.9	3.47
B	1/2	1.376	2.74	0.821	11.0	4.01	2.94	1.391	1.76	56.8	4.24
C	1	4.720	64.4	6.54	7.44	0.116	1.64	4.674	4.03	300	264
C	1/2	4.500	29.3	2.00	2.50	.0852	2.51	4.677	8.41	292	234
D	2	1.300	9.80	1.46	21.9	2.24	3.89	1.318	2.68	86.3	5.76
D	1	1.320	5.04	0.801	11.6	2.31	2.91	1.338	2.33	108	7.44

CONCLUSIONS

The steady-state forced vibration tests described in this paper have shown how this technique can be used fairly quickly to give accurate assessments of the dynamic characteristics of real structures in both horizontal directions as well as in torsion.

The natural frequencies and therefore the modal stiffness of all the modes fell after the first pseudo-dynamic test. However, this was much more pronounced in the fundamental east-west mode where the reduction in frequency was about 45%, than in north-south direction where the reduction was only about 25%. The equivalent figure for torsion was 35%. There was an increase of about 30% in the damping in all three modes.

Adding the masonry infill panels had little effect in the north-south direction, the natural frequency of the fundamental mode and the modal mass increasing by about 10%. However, in the east-west direction the frequency of the fundamental mode increased from 0.96 Hz to 3.8 Hz, and there was an almost fivefold increase in the modal mass, producing a modal stiffness about 70 times higher. There were similar increases in the natural frequency, modal inertia and modal stiffness in the fundamental torsional mode.

The frequencies of all the fundamental modes fell again, to about the values obtained before the infill panels were built, after the further pseudo-dynamic tests and the removal of the panels at the ground floor level. However, as the frame underwent two different pseudo-dynamic tests and had some panels removed between this forced vibration test and the previous one, it is not possible to say how much of this reduction was attributable to each of these events.

ACKNOWLEDGEMENTS

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