ORIGINAL PAPER

# Vibration monitoring and condition assessment of the University of Sheffield Arts Tower during retrofit

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**Abstract** The 78 m Arts Tower at the University of Sheffield was refurbished during the period 2009–2011, with improvements that included replacement of façades and partitions. The structural changes were reflected in changes in dynamic properties that were tracked by a combination of long-term acceleration monitoring and vibration tests to identify local and global vibration modes. The global horizontal plane modes of the building were correlated with finite element model simulations and real-time automated operational modal analysis to characterize the effects of the structural changes on horizontal vibration modes in sway and torsion. In addition, floor vibration tests linked to the finite element simulations identified the contribution of 'non-structural partitions' on the floor dynamic performance. Replacement of heavy masonry partitions with modern lightweight forms has resulted in reduction of stiffness, yet there is no obvious change in sway vibration serviceability and floor performance remains satisfactory.

**Keywords** Building · Vibration · Monitoring · Retrofit · Floor · Sway · Partition

## 1 University of Sheffield Arts Tower history, structural details and upgrade

The University of Sheffield Arts Tower has the distinction of being the tallest university building in the UK and until 2009 was the tallest building in Sheffield. The building was

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completed in 1965 and is now a Grade II listed property, in recognition of its heritage status and imposing strict requirements on structural alterations including a retrofit that was completed in 2011.

The building (Fig. 1) has 21 storeys above ground floor (level 0), 2 basement levels, and is approximately 80 m high, 36 m wide and 20 m deep. The basement houses lecture theatres and plant rooms which extend outside of the tower footprint and level 1 is 8.54 m above level 0, with a mezzanine level. Normal floors are spaced at 3.55 m intervals and as built were sub-divided by non-load bearing 75 mm thick 'breezeblock' masonry partitions

The structural frame comprises 250 mm reinforced concrete (RC) flat slabs spanning between two closely spaced RC cores and RC columns arranged around the perimeter of the floors (Fig. 2). There are two frame arrangements; below level 1 there are 16 perimeter rectangular columns of 965 mm × 965 mm, with a transfer slab to the 94 rectangular upper columns of 203 mm by 406 mm at 1.8 m centres in the long direction and 1.12 m in the short direction.

Structural lateral stability is principally provided by the two reinforced concrete shear cores acting as vertical cantilevers. Lateral load is transmitted to the cores through the building cladding into the edge of each floor and then transferred by membrane plate action to the core walls. The cores are offset with regard to the floor plate long axis dividing floors into narrow and wide sides. Because of the structural eccentricity, the building's response under lateral loads involves both lateral and rotational displacement.

Shortly after the Arts Tower was first occupied in 1966, a number of occupants reported perceiving movements of the upper floors, and this prompted a vibration study by Building Research Establishment (BRE) [3, 8]. The study concluded that the significant contribution of the non-structural components such as partitions, walls and

**Fig. 1** Views of Arts Tower: during retrofit (*left*), after retrofit (*middle pair*) and east elevation

cladding to lateral stiffness prevented a rather less satisfactory performance in wind. It was also noted at the time that the masonry partitions, which were not designed to resist shear loads, had diagonal cracks on the upper floors. Their involvement in resisting wind-induced vibration was believed to have caused this cracking.

Due to changing occupancy requirements, the building was retrofitted between 2009 and 2011 (http://www.sheffield.ac.uk/artstowerproject) in three phases, the first two being relevant for structural performance. In view of the effect of the partitions observed in the earlier studies, the effect of their removal was a particular concern to the consultant, who carried out their own structural analysis and commissioned performance monitoring during the retrofit.

Because of the current uncertainty concerning the effect of partitions on floor vibrations and global sway for a broad class of buildings (partitions are assumed not to contribute), the opportunity was taken to conduct a research project on building dynamic behaviour to run concurrently with the monitoring.

For the retrofit, the plan was that in Phase 1 the external façade/curtain walling would be replaced from the first floor to the crown, and lifts (including an unusual paternoster lift) would be refurbished. Levels 9–19, badly needed by the University's Architecture Department were to be refurbished first. Then in Phase 2, levels 1–8 would be refurbished. Refurbishing a floor would entail removing all partitions and installing a mixture of fixed stud (metal and plasterboard), fixed glass and retractable partitions.

Figure 1 shows the building at various stages of the operation; with original façade and following its replacement, while Fig. 2 shows the internal floor plan before and after the retrofit and Figs. 3 and 4 show the original and replacement partitions. The original heavy masonry partitions have been largely demolished in the lower left view in Fig. 3 which shows the narrow side of the floor. The middle view shows a totally cleared floor on the wider side with the two cores on the left, and the lower right view shows one of the many partition arrangement in the new configuration. Some floors now have a combination of the fixed and sliding partitions, while open plan areas have half-height 'cantilever' partitions and offices with glass partitions.

## 2 Vibration studies and the contribution of partitions

To date, there are no published studies comparing sway performance of a tall building with and without partitions both experimentally and analytically. Several studies have considered the effects of partitions and internal walls through finite element simulation and comparison with performance of a building only in the state of having the partitions present. These include apartment blocks [11, 15] and an office tower [6]. All these studies point to the stiffening effect of non-structural elements that plays a role in the low-level vibration response.

There has also been a body of research on the contribution of non-structural elements, including partitions, to the dynamic characteristics of floors in relation to vibration serviceability. The most recent contribution by Setareh [13] points out that most of the findings and code guidance relate to the possible increase in floor damping due to partitions, with little hard evidence on the contribution to stiffness. Some evidence on the stiffness contribution is available from
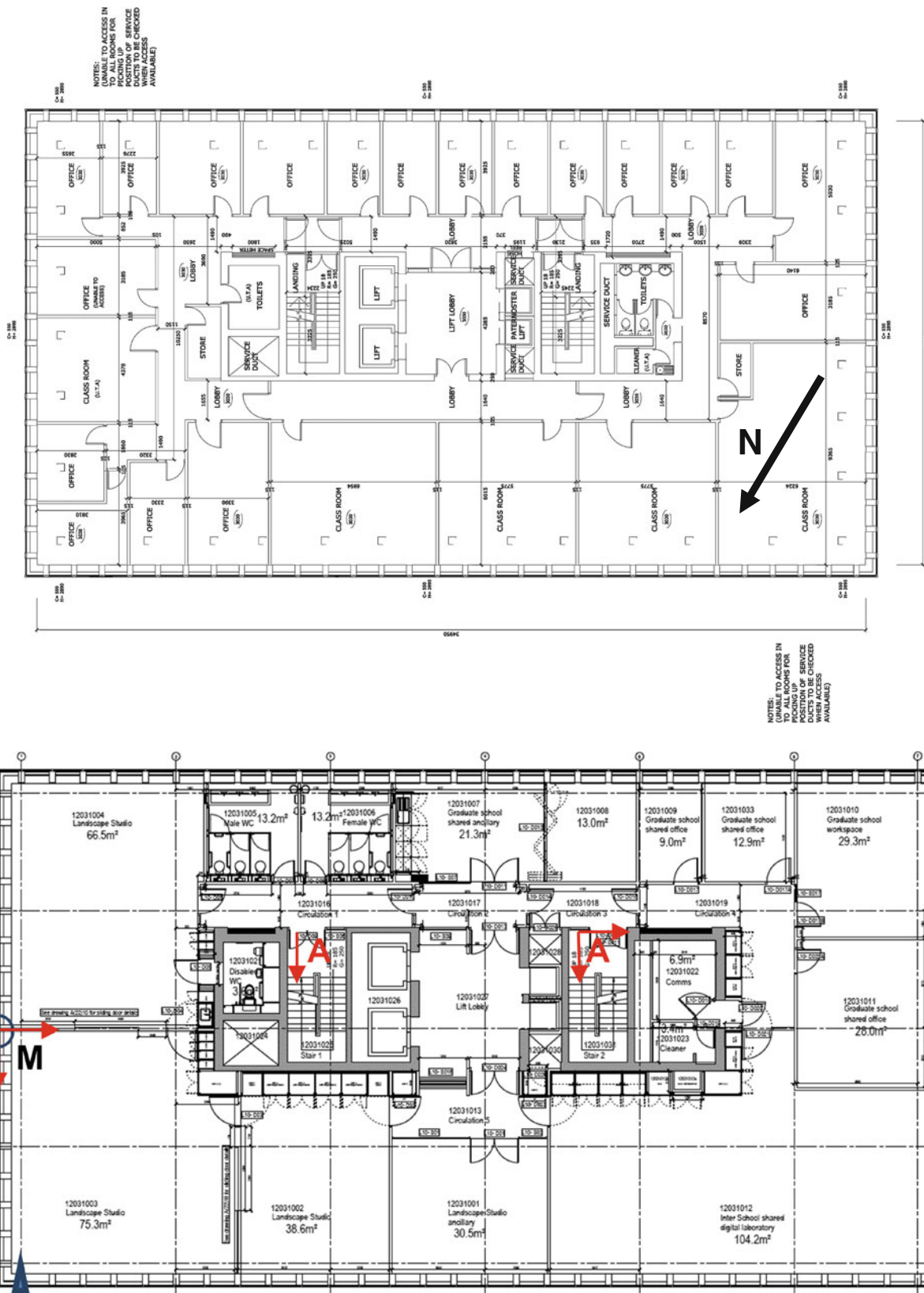
**Fig. 2** Typical floor plan (example for level 10) showing two cores and partitioned office spaces before (*upper*) and after (*lower*) retrofit. *Triangle* (*lower left*) matches origin of measurement grid (Fig. 11).

*M* marks location in plan and orientation of rooftop-mounted accelerometers used for long term monitoring, *A* marks location in plan and orientation in plan of accelerometers used for ambient vibration testing

**Fig. 3** Half-demolished masonry partitions, cleared floor and lightweight steel frames for new fixed partitions



**Fig. 4** Post-retrofit partition arrangements. Level 7 (*left*) full-height glass and half-height cantilever, level 8 (*middle*) half-height only and level 18 (*right*) movable partitions retracted, with hammer test in progress

laboratory mockup one-way spanning floors reported by Falati [7], whereas systematic updating of a finite element model of an as-built multi-partitioned floor by Miskovic et al. [9] was linked to the considerable influence of partitions usually assumed in vibration serviceability assessments to have no effect. In both cases, the partitions appeared to work like vertical springs; in the latter case, per-metre stiffness values were derived for both plasterboard and glass partitions to obtain best-fit of experiment and FE modeling.

As a contribution to this limited but useful body of research, this paper describes dynamic evaluations through measurement and simulation having the aim of identifying the contribution of the 'non-structural' partitions to global and local dynamic properties and their influence on vibration serviceability.

Although vibration levels until the retrofit have apparently remained within recognised tolerances for occupants, the removal and subsequent replacement of the partitions led to concerns about possible adverse effects on vibration performance that were addressed through prior analytical studies by the consultant (Gifford) and checked by limited vibration monitoring.

To supplement the consultant's monitoring, Full Scale Dynamics Ltd (FSDL), a spin-out of the University of Sheffield was commissioned to monitor global horizontal

vibration performance of the building during the retrofit. In addition, a series of brief vibration measurements and subsequent finite element (FE) analyses of global (horizontal building sway) and local (vertical floor vibration) corresponding to stages in the retrofit were carried out by Vibration Engineering Section (VES).

This paper first describes the brief vibration measurements (by VES) and extended monitoring (by FSDL), focusing on changes in the modal properties and dynamic performance. Since they came last in the chronological sequence, the FE modelling and attempts to explain the observed performance changes are described last. Refinements of the FE models, stopping short of formal model updating, are described, leading to conclusions about effects of non-structural partitions on both global and local dynamic performance.

## 3 Vibration measurements

A series of vibration measurements have been used to characterize the global (sway) and local (floor, vertical) dynamic behaviour of the building.

For the sway behaviour two modal (vibration) surveys of the building have been conducted to identify the

horizontal plane vibration modes; in 1987 by the BRE using artificial excitation and in 2011 by VES using ambient vibration. Two long-term monitoring exercises have also been carried out spanning the complete retrofit period. In total, there is enough information to provide a clear picture of the effect of the retrofit on horizontal modal properties.

For the local (floor) dynamics, forced-vibration tests of sample floors were carried out in 2009 and 2011 to characterise the vertical vibration modes likely to be excited by pedestrian occupants. While it was not possible to carry out exactly equivalent measurements before and after retrofit, there is sufficient information to identify changes in floor vibration properties for the different partition arrangements.

### 3.1 BRE forced-vibration test (horizontal modes), 1987

The first vibration study of the building was a 9-day campaign by BRE in July 1987 [8]. Four 'rotating eccentric mass' (REM) shakers manufactured by University of Bristol and each capable of generating up to 1.05 kN amplitude uniaxial harmonic force were used to excite building response in single modes at vibration levels similar to those likely to be experienced in strong winds. The step sine approach was used, incrementing excitation frequency at 0.1-Hz interval from 0.1 to 10 Hz. The shakers have a $mr\omega^2$ force amplitude characteristic for fixed radius of gyration $r$ for the pair of contra-rotating eccentric armatures with total mass $m$ having angular velocity $\omega$. Shaker force was calculated but not measured directly, and without phase information natural frequencies and damping ratios were estimated by curve-fitting to ratio of response amplitude to exciting force. Highly accurate

frequency and damping estimates were obtained from the free decay of single modes at resonance after halting the shakers. Mode shapes were obtained using relative amplitudes and phases between a pair of accelerometers: one a fixed reference, the other roving to locations at all 23 levels of the building.

Modal parameters for modes identified by BRE are summarised in rows 2 and 3 of Table 1, for the largest shaker force ('high'), producing single-mode amplitudes of 1.2 mm, and using 20 % of full force to generate response that would be associated with modest wind speeds ('low'). Higher response levels were shown to result in lower frequencies, and also in higher damping ratios, as observed in the more recent monitoring. This table also reports the results from subsequent exercises, for comparison. The long (36 m) axis of the building is the more closely oriented to the East–West direction (Fig. 2), hence, the lateral sway in the direction of the short (20 m) axis is labeled 'NS'. Due to the asymmetry of the cores, translational motion is accompanied by varying degrees of rotation, but modes which appear to have a centre of rotation within the building plan are labeled $\theta$ (TH).

### 3.2 VES ambient vibration survey (horizontal modes), June 2011

Rotating eccentric mass shakers are now relatively uncommon for testing civil structures due to severe logistical and safety constraints, a notable contemporary example being the NEES facility based in Los Angeles Stewart et al. [14]. Because heavy equipment is not required, the ambient vibration survey (AVS), coupled with operational modal analysis (OMA) is now a standard procedure.

**Table 1** Observed mode frequencies for horizontal vibration modes by experimental modal analysis (EMA), 1987–2012

| Mode | NS1 | θ1 (TH1) | EW1 | NS2 | θ2 (TH2) | EW2 | θ3 (TH3) | NS3 | EW3 |
|---|---|---|---|---|---|---|---|---|---|
| 7/1987 (high) | 0.674 | 0.779 | 0.845 | 2.368 | 2.427 | 2.800 | 4.370 | 5.67 | 6.56 |
| 7/1987 (low) | 0.688 | 0.790 | 0.856 | 2.424 | 2.480 | 2.835 | 4.380 | 5.71 | 6.6 |
| 1/2009 (high) | 0.620 | 0.740 | 0.80 | 2.350 | – | 2.730 | – | – | – |
| 10/2009 (low)[a] | 0.626 | 0.752 | 0.806 | 2.157 | 2.197 | 2.677 | – | – | – |
| 8/2010 (low)[b] | 0.610 | 0.720 | 0.770 | 2.240 | 2.270 | 2.680 | – | – | – |
| 11/2010 (high)[c] | 0.540 | 0.670 | 0.720 | 2.200 | 2.260 | 2.630 | – | – | – |
| 3/2011 (high) | 0.530 | 0.670 | 0.720 | 2.120 | 2.190 | 2.600 | – | – | – |
| 6/2011 (low) | 0.556 | 0.688 | 0.725 | 2.160 | 2.220 | 2.630 | 3.980 | 4.52 | – |
| 6/2011 test[d] | 0.564 | 0.686 | 0.736 | 2.160 | 2.220 | 2.630 | | | |

*Low* weak ambient vibration, *high* with strong winds or shaking

[a] During early stage of Phase 1 with most partitions present

[b] With levels 1–8 partitions present

[c] All partitions removed

[d] Final partition arrangement after retrofit
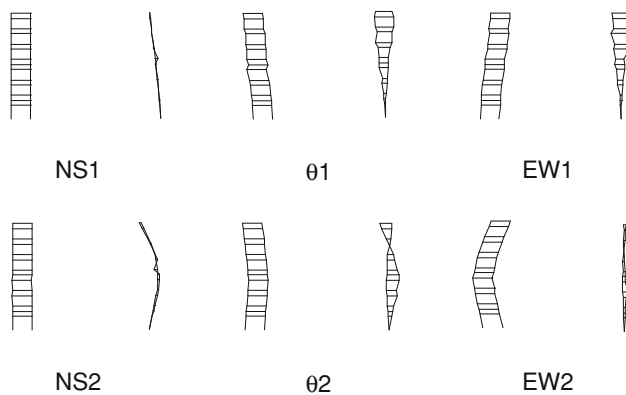
**Fig. 5** Mode shapes from AVS, June 2011 for orthogonal elevations in long axis and short axis

As part of a student Masters level research project [2] that involved modeling and testing the horizontal modal properties of the building, an AVS of the building was carried on the afternoon of 10 June 2011, using 24 accelerometers (type: QA750). On two reference levels (levels 18 and 8), two accelerometers were aligned in the NS direction in the east and west stairwells, with a third aligned in the EW direction; the plan-wise locations are shown in the lower view of Fig. 2. The remaining 18 accelerometers were arranged at six other levels for a single recording of 16 min. The 18 accelerometers were then moved to different floor levels for another 16-min recording. Due to time constraints, every third (non-reference) floor was missed, which affects appearance of mode shapes.

Subsequent OMA using the Eigen-system realisation algorithm (ERA) implemented in MODAL software [4, 5] identified the 2D characteristics of modes up to 15 Hz. As the mode shapes are not expected to have varied significantly during the retrofit, they are presented as a reference set in Fig. 5, for the first six modes viewed in the EW and NS directions, with the frequency estimates for 10 June

2011 given in the last row of Table 1. NS1 and NS2 are pure bending due to the symmetry of the building whereas EW modes are less well aligned and involve torsion due to the asymmetry of the core locations.

Only the first six modes can be readily compared with the BRE results. Clearly the mode frequencies have dropped significantly since 1987, which reflects the effects of the retrofit as well as any degradation taking place over the intervening years.

### 3.3 Vibration monitoring (horizontal modes), 2009–2012

From the beginning of the retrofit period, a monitoring system was operated by Strainstall. Alas, because the recordings were event triggered and effectively randomly sampled in time it was not possible to identify definitely the modal frequencies immediately preceding the retrofit. In September 2009, a little too late to catch the very start of the retrofit, a vibration-monitoring system was installed by FSDL and operated until March 2012. The system comprised a pair of Guralp CMG-5TD triaxial accelerometers synchronised by GPS and mounted at two locations on the roof, in plan-wise locations indicated in the lower view of Fig. 2. Power spectral densities (PSDs, in square-root form) of response in the short axis (N) and long axis (E) for 4 January 2011, due to moderate winds are shown in Fig. 6. Mode NS1 (0.56 Hz) only appears in the N response, while modes NS2 and θ2 (2.16 Hz/2.22 Hz) are merged into one peak by the averaging process and the small variations of modal frequency during the day.

All the processed data were stored in a database running on the FSDL server using MySQL 5.1. Temperature and wind speed and direction data were estimated by averaging meteorological data accessed via http://weather.noaa.gov from three weather stations around Arts Tower: Manchester Airport, East Midlands Airport and Humberside

**Fig. 6** Broadband displacement power spectra for 1 h of monitoring, 4 January 2011
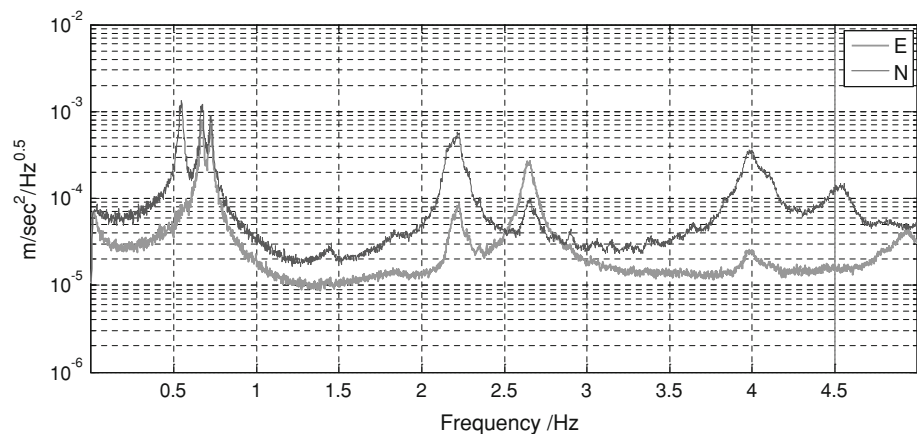
**Fig. 7** Frequency variation in sway modes. P1S and P1E mark start/end of Phase 1 partition removal, FE marks end of removal and replacement of façades, P2S, P2E mark start/end of Phase 2 partition removal
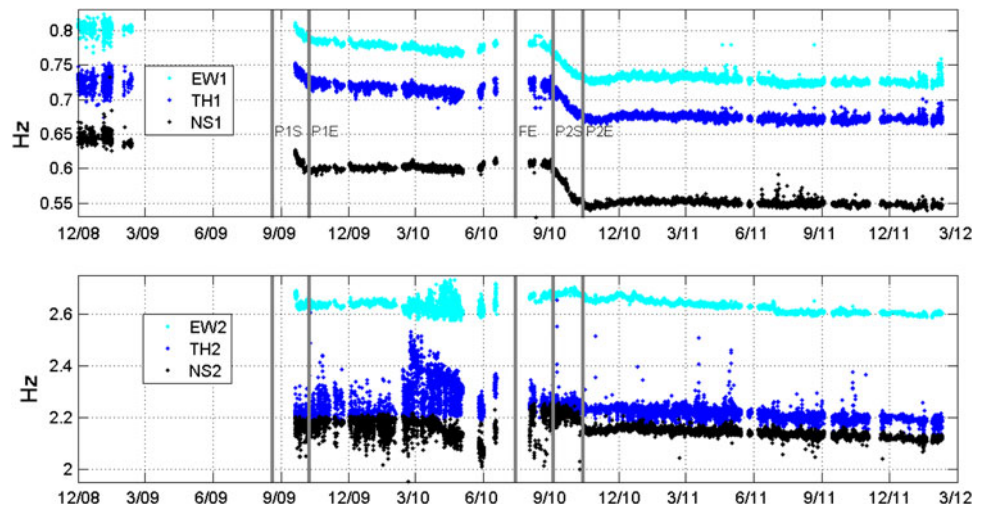


Airport. Using the previous 30 min of data, automated processes running on the server calculated RMS accelerations and estimated modal parameters (frequency, damping and mode shape ordinate) using the stochastic subspace identification (SSI) technique [12].

Figure 7 shows variation in natural frequencies for the lowest six modes for the duration of the FSDL monitoring. Frequencies for the first three modes were also estimated from the Strainstall monitoring data corresponding to several months before the retrofit, and due to the short event-triggered recordings, the same SSI process results in larger variation than the later estimates. There are several notable features in the data which are discussed in a later section.

### 3.4 Floor vibration testing (vertical modes), 2009 and 2011

These measurements were undertaken within two Masters student projects, a study in 2009 [10] of the floor dynamics of cleared and uncleared areas and one in 2011 [1] focusing on modeling and testing the post-retrofit floor behaviour. The vertical mode properties of sample floors were studied experimentally during the retrofit for floors with and without original partitions in place and post-retrofit for open plan and lightly partitioned flows as follows:

- Two rooms of level 8 with full masonry partitions above and below were tested (using an instrumented hammer) in June 2009. This condition is referred to as L8_2009.
- An area equivalent to the partitioned level 8 area but at level 15 was tested (using instrumented hammer) in June 2009. For this level, the partitions above and below had been completely cleared (Fig. 2 left). This area is referred to as L15_2009.

- The wide bay of the post-retrofit repartitioned level 8 (Fig. 4 middle) was tested (using an electrodynamic shaker) in June 2011. The area tested included the area tested in 2009, but at lower resolution, i.e. twice the spacing of measurement points. This area is referred to as L8_2011.
- A strip of the wide bay of level 18, being a subset of the level 8 area tested in 2011, was tested (using an instrumented hammer) in June 2011 with movable partitions extended (and engaged) and retracted (Fig. 4 right), but with no change to the partitions below on level 17. This area is represented as L18_2011P and L18_2011.

Figure 11 summarises the measurement areas for the various floor conditions, with the grid covering the wide bay of the floor and areas adjacent to the cores. The rectangular grid of dots represents the measurements points used in L8_2009 and L15_2009 testing resampled to match the L8_2011 measurement grid. The line of dots represents the L18_2011 measurement poi. The measurement point numbers are referred to in the figures that follow, particularly points 21–24.

Floor vibration test procedures used either roving instrumented hammer and fixed accelerometers or fixed electrodynamic shaker excitation and roving response measurements, in all cases using Honeywell QA 750 servo accelerometers. Modal analysis used the global rational fraction polynomial method in the MODAL software [4, 5].

## 4 Influences on vibration performance

With the comprehensive datasets from the AVS, monitoring and floor vibration testing, it is possible to examine the changes in modal properties not only due to the retrofit but
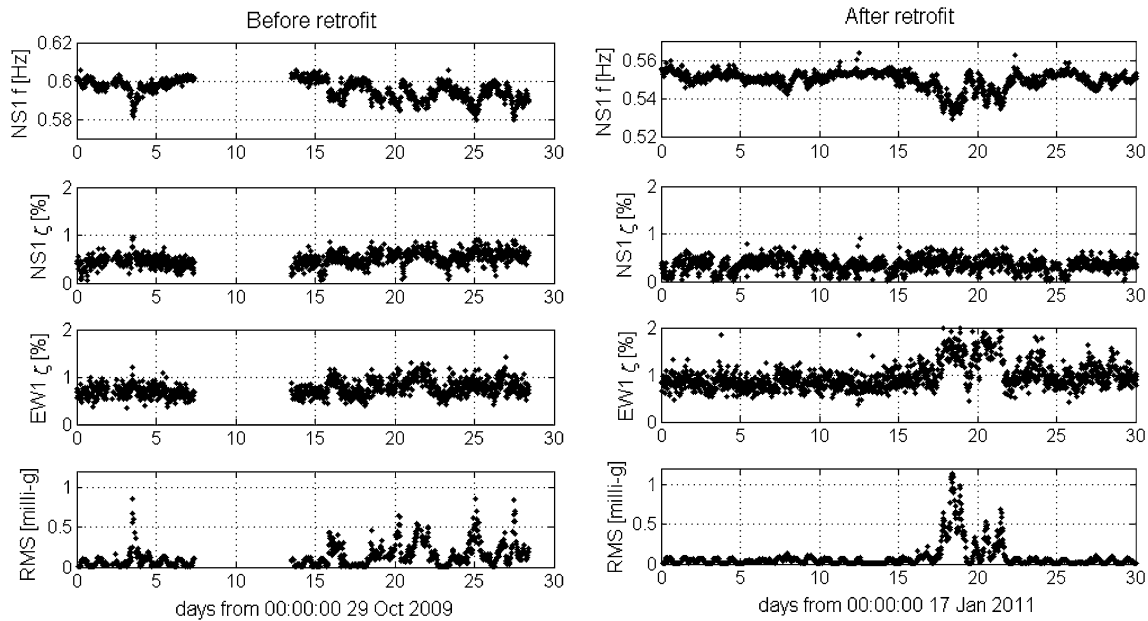
**Fig. 8** Modal properties and response levels before and after Phase 2

### 4.1 Global horizontal mode properties

The most remarkable feature demonstrated from the monitored frequencies in Fig. 7 is the two periods during which the frequencies of the first three horizontal vibration modes (NS1, θ1, EW1) dropped rapidly. These correspond to the two distinct phases of partition removal.

Phase 1 removal of masonry partitions (levels 9–19) began in late July 2009 (before monitoring was operational) and was completed on 9 October 2009. This corresponds to the first period of steadily decreasing frequencies.

Removal of the original façade was carried out from Sept 2009 to February 2010 working from the top (level 19) down. Installation of new façade was carried out from November 2009 to July 2010 working from the top down. The early part of the façade removal overlaps the partition removal, so a distinct effect cannot be seen, however, the rise in frequencies over mid-summer 2010, even with much missing data, corresponds with the façade re-installation.

Phase 2 removal of masonry partitions (levels 8 down to level 1) began in the first week of September 2010, with 1 week per floor. This period corresponds to the fastest and most significant drop in frequencies in Fig. 7.

The set of second horizontal modes (NS2, θ2 and EW2) displays a rather different pattern to the set of first modes (NS1, θ1 and EW1). The changes are proportionally smaller, in particular, the dramatic reduction in the first-mode frequencies is replaced for the second modes by a modest rise followed by a modest fall, reaching a minimum frequency simultaneously with the first-mode frequencies. From then until the last data, second-mode frequencies drop steadily, and more significantly (even in % terms) than the first-mode frequencies.

Figure 8 zooms in on 1-month periods before and after Phase 2 showing variation in NS1 frequency, of NS1 and EW1 damping and of 30-min RMS acceleration, for vector sum of broadband E and N components from a single roof level accelerometer. In both cases, maximum 30-min RMS response exceeded 1 milli-$g$ (9.81 mm/s$^2$), which corresponds to 0.5 mm for a frequency of 0.7 Hz. Instantaneous values, estimated by double-integration of acceleration data, reached 5 mm, larger than (1.2 mm) single-mode amplitudes generated by BRE shakers.

NS1 frequency shows approximately 0.02 Hz reduction per milli-$g$ of response, EW1 similar, but the damping variation is more interesting. Before Phase 2, EW1 and NS1 show similar ranges consistent with the BRE results and modest increase with stronger response. After Phase 2, NS1 damping range has dropped while EW1 shows very clear dependence on amplitude. Histograms for the full durations of FSDL monitoring before and after Phase 2 are shown in Fig. 9, clarifying the changes.

With reduced frequency-implying reduced stiffness and altered damping ranges, what is the impact on response levels? There are very few reliable data points from processed 30-min records to demonstrate convincing correlations, so the alternative approach of extreme value statistics

**Fig. 9** Damping ratios before and after Phase 2
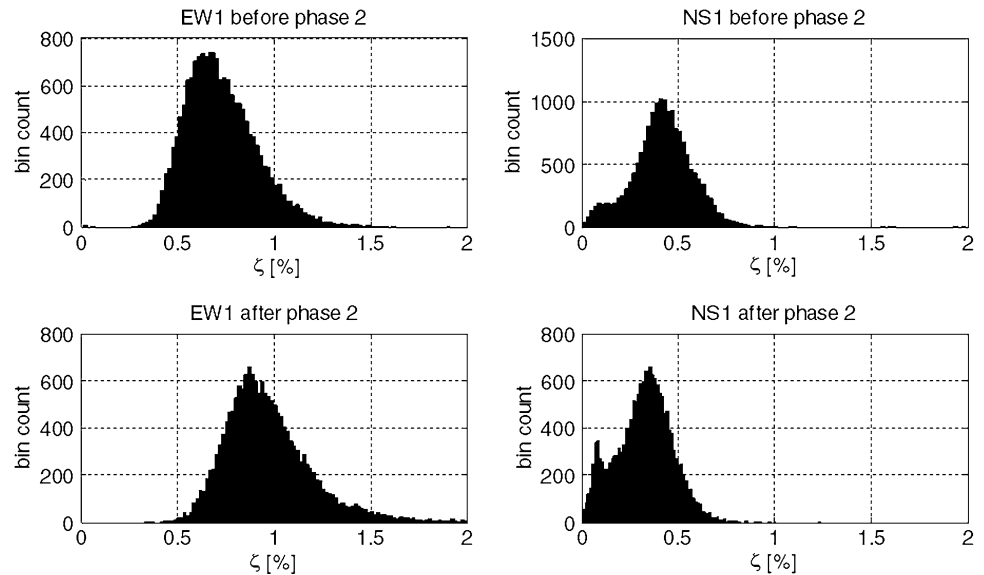


is applied. The daily RMS maxima are obtained and their rank plotted as 'reduced variates' against corresponding response level as a Gumbel plot in Fig. 10. While the maxima do not follow a linear trend (which would indicate a Gumbel distribution), the distributions are remarkable similar, strongly suggesting that Phase 2 alterations had no significant effect on response levels.

A possible explanation is that the increased EW1 damping, proportionally more than the NS1 reduction, offset the reduced stiffness.

### 4.2 Floor vertical mode properties

#### 4.2.1 Comparison of 2009 cleared and uncleared floors (L8_2009 vs. L15_2009)

From L8_2009 and L15_2009, it is possible to compare modal frequencies of different partitioned and un-partitioned levels, but as the floor slabs are only nominally similar, the value of the comparison may be limited. Comparison is made using frequency response functions (FRFs) representing the ratio of harmonic acceleration response at a point $j$ to harmonic force at a point $i$. Rather than attempting comparison of like-for-like FRFs (same $i$–$j$ pairs, not possible across the full set of measurements at all stages and floors), the strengths and peak frequencies of the FRF peaks are compared for points 21–24 and other nearby points as indicated in Fig. 11. For these two measurements, the response points were fixed and the hammer roved (labeled 'b' in Fig. 11).

Figure 12 shows the FRFs for L8_2009, while Fig. 13 shows FRFs for L15_2009, using the same reference point. Two things are immediately obvious when comparing the two floors.
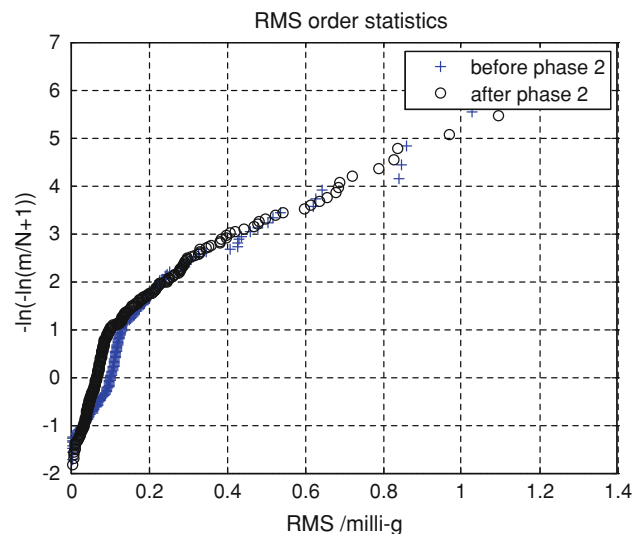


**Fig. 10** Gumbel plot of rank (m) of daily maximum RMS values before and after Phase 2

First, the peaks for (cleared) L15_2009 are much sharper than those of L8_2009, which shows a lower damping, and the strongest FRF below 20 Hz is 50 % stronger for the cleared floor. Hence, the partitions have provided additional damping.

Second, the first two clear frequencies for L15_2009 are shifted down compared to L8_2009, the obvious conclusion being that the partitions provide considerable stiffness but relatively little mass.

#### 4.2.2 Comparison of level 8 before and after retrofit (L8_2009 vs. L8_2011)

Due to the sequence of partition removal it was not possible to investigate level 8 in the same condition as level

**Fig. 11** Floor vibration test grids. *Larger grid* represents level L8_2011 test grid (points spaced in line with every second column), *line of dots* represents L18_2011. *Large dots* represent reference points for (*left to right*) measurements on: **a** level 18; **b** level 15 and level 8 (during retrofit); **c** level 8 (post-retrofit)
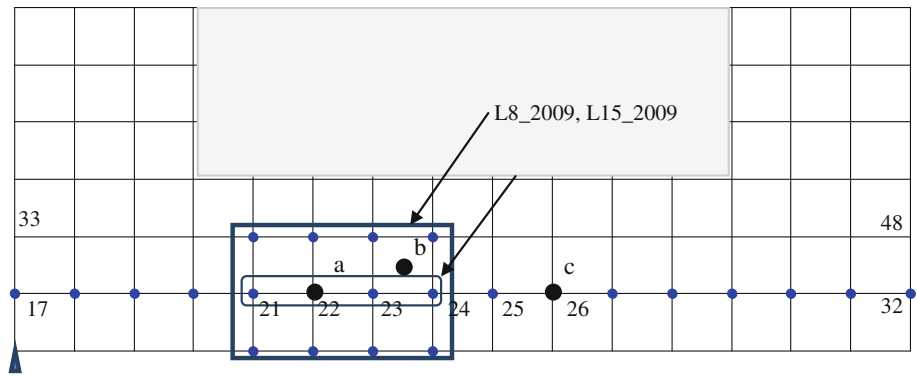
**Fig. 12** L8_2009 FRFs: un-cleared level 8 points 21–24. Note three modes up to 16 Hz
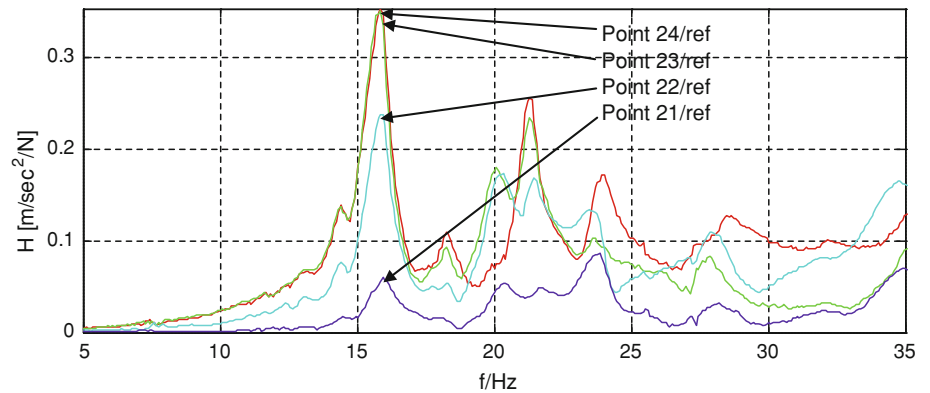
**Fig. 13** L15_2009 FRFs: cleared level 15 points 21–24; ref point is closes to points 21 and 22
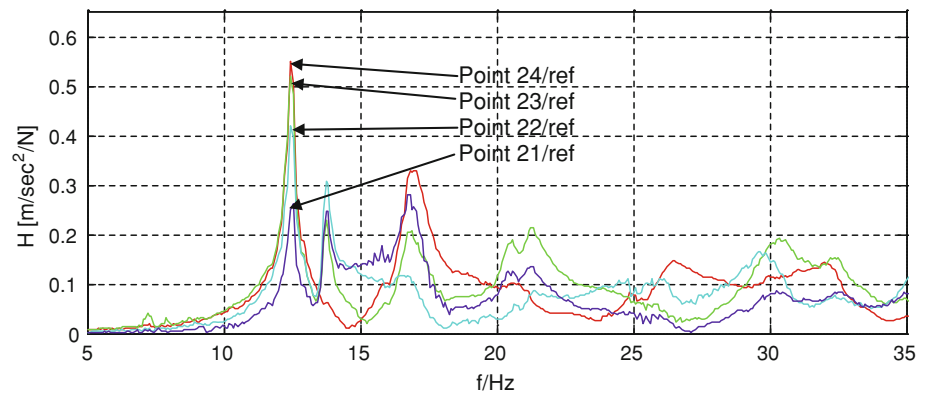
15, i.e. completely cleared during the retrofit. However, for this level post-retrofit, there are no full-height partitions above and below the area tested (only the cantilever partitions) so it might be expected to behave as if cleared above and below. Only partial mode shapes were obtained in 2009, compared with the detailed shapes obtained in 2011, and the reference measurements points were different but it is still possible to make valid comparisons via the composite FRFs of Figs. 12 and 14. There appears to be a downward shift of mode frequencies from L8_2009 to L8_2011 but the character of the two FRF sets are rather different. Probably the first three modes (to 16 Hz in L8_2009 and to 14 Hz in L8_2011) correspond; however, beyond that, the few matching point measurements are not

enough to make a convincing comparison using, for example, modal assurance criterion. Unlike the L8_2009 to L15_2009 (uncleared vs. cleared, but different levels) there is no obvious change in either sharpness of the peaks or FRF maxima before and after retrofit for level 8, i.e. no conclusive effect on damping.

### 4.2.3 Effect of movable partitions (L18_2011)

From the level 18 measurements, the effects of the movable partitions could be observed. Figure 15 shows point mobility FRFs ($i = j = 22, 26$) for two locations with and without partitions engaged over the remaining half of the floor above level 18. The engagement is effected

**Fig. 14** L8_2011 FRFs: cleared level 8 (points 21–26 including point mobility). Note three modes up to 14 Hz
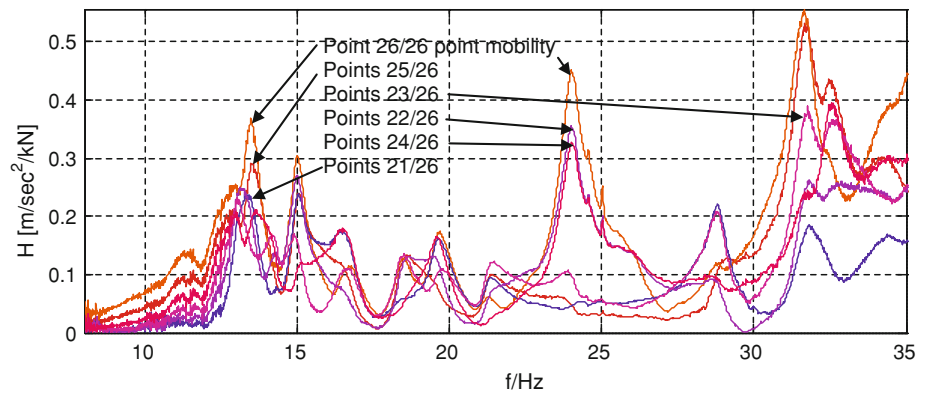


**Fig. 15** L18_2011: point 22 and 26 driving point mobility FRFs with or without extended partitions
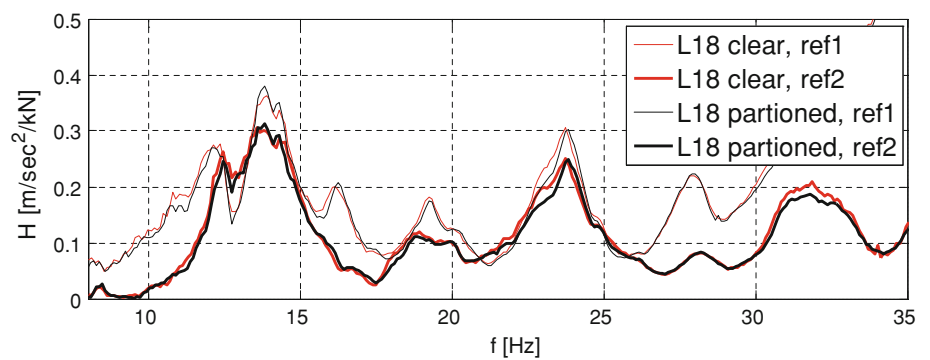


by forcing out a set of locking pins to push up and down but there is negligible effect on the FRFs; for both locations, the FRFs overlay perfectly with or without partitions extended.

### 4.2.4 Mode shapes for L8_2011

Mode shapes estimated for the comprehensive post-retrofit level 8 measurements are shown in Fig. 16. The first three modes follow an expected pattern of global deformation with increasing order along the bay and all except the first mode are clearly visible in the FRFs of Fig. 14. There is no second-order mode in the short direction (across the wide bay) below 30 Hz.

## 5 Finite element modeling to simulate modal performance

According to the measurements, the internal rearrangement of the partitions has had significant influence on global and local dynamic behaviours. To study the effect of the partitions and their removal on global (sway) and local (floor vibration) behaviour of the Arts Tower, finite element models were created with ANSYS software using BEAM44 elements to represent columns and beams

(perimeter and transfer) and SHELL63 for slabs, core walls and partitions. Several models were developed to simulate the building at various stages of the retrofit:

*Model 1* It is shown in Fig. 17 and represents the final state of the Arts Tower after removing partitions and including main structural elements (columns, slabs and core walls).

*Model 2* It is based on Model 1 and represents the state of the Arts Tower after the first partition removal phase (removal of partitions at levels 9–19).
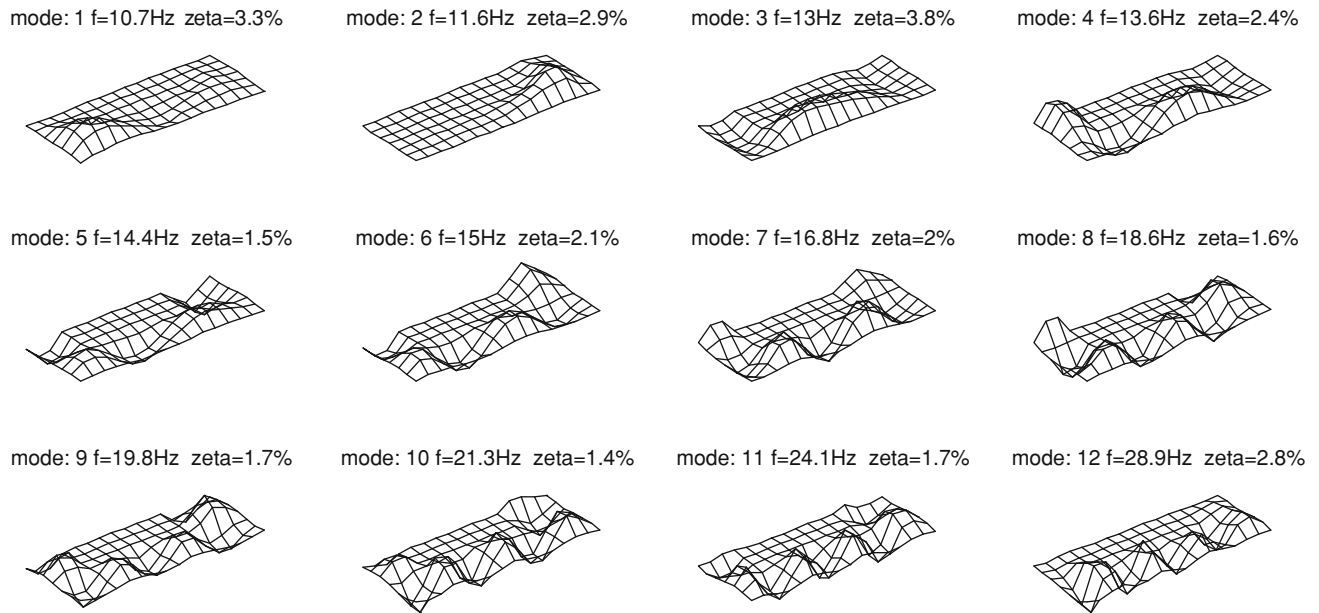
*Model 3* It is also based on Model 1 and represents the original building before the retrofit, i.e. with all partitions in place.

*Model 3 variants*: are as Model 3, but with changes to details of partitions and other structural components.
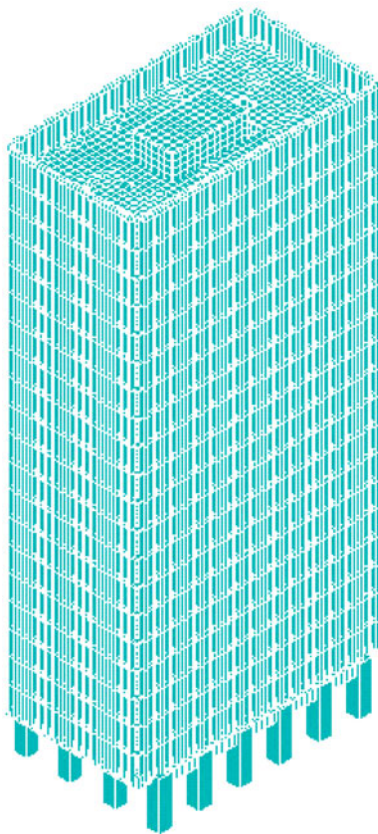
*Model 4*: represents a slice of the building at level 8 with structural components above and below to model the behaviour of this level in detail.

Several assumptions were made in the modelling.

First, based on compressive tests on cores from different levels that showed a large range of strengths, a value of $E = 32.5$ GPa was taken for the dynamic Young's modulus.

mode: 1 f=10.7Hz zeta=3.3%    mode: 2 f=11.6Hz zeta=2.9%    mode: 3 f=13Hz zeta=3.8%    mode: 4 f=13.6Hz zeta=2.4%

mode: 5 f=14.4Hz zeta=1.5%    mode: 6 f=15Hz zeta=2.1%    mode: 7 f=16.8Hz zeta=2%    mode: 8 f=18.6Hz zeta=1.6%

mode: 9 f=19.8Hz zeta=1.7%    mode: 10 f=21.3Hz zeta=1.4%    mode: 11 f=24.1Hz zeta=1.7%    mode: 12 f=28.9Hz zeta=2.8%

**Fig. 16** Mode shapes for L8_2011, i.e. level 8 after retrofit

**Fig. 17** Finite element model for sway, slice at level 8 and partitions with openings at level 1

Second, rigid fixity was assumed at all connections and the columns were assumed fixed at base, with no basements modelled and no soil–structure interaction.

Third, 0.1 m thick partitions that included plaster cover were assumed to be fixed at the slabs with 1,400 kg/m$^3$ density. A value of 3 GPa for Young's modulus was initially used, in line with the consultant FEM but it became immediately apparent that a value of 1 GPa provided a much better matching of both floor and sway vibration modes, so the starting point here is the 1 GPa simulation set.

Fourth, despite some variation of partition arrangement from floor to floor, partitions for each floor in Model 2, Model 3 and its variants were modelled as they exist on floor 1, while accounting for major differences.

The sole aim of the modelling was to obtain Eigen-solutions representing the mode frequencies and shapes for global (sway) and local (floor vibrations) behaviour, and these solutions are discussed for the various models in relation to the observed modal properties. The comparison is focused on the first and second sets of three modes in the horizontal plane which might respond most strongly to wind loads, and the floor modes up to 25 Hz which might respond most strongly to footfall loads.

### 5.1 Models 1–3 characterising sway modes

Table 2 summarises the frequency matching between Models 1–3 and the corresponding measurements. Visual comparison of the FEM modes (for the three models) and the experimental modes expanded to the same geometry (not presented here) indicates that the model reflects well enough the character of the full-scale modes and that the mode shapes do not change perceptibly among the models.

**Table 2** Matching of measured horizontal mode frequencies and analytical estimates for Models 1–3, using 1GPa partitions

| Mode | Model 3 frequency (Hz) all levels partitioned | | | Model 2 frequency (Hz) levels 1–8 partitioned | | | Model 1 frequency (Hz) no partitions | | |
|------|-------|------------|---------|-------|------------|---------|-------|------------|---------|
| | FEM 1 | EMA 10/2009 | % Error | FEM 2 | EMA 8/2010 | % Error | FEM 3 | EMA 11/10 | % Error |
| NS1 | 0.647 | 0.626 | +3.3 | 0.609 | 0.61 | 0 | 0.523 | 0.54 | −3.1 |
| θ1 | 0.758 | 0.752 | +0.7 | 0.737 | 0.72 | +2.3 | 0.636 | 0.67 | −5 |
| EW1 | 0.856 | 0.806 | +6.2 | 0.854 | 0.77 | +10.9 | 0.751 | 0.72 | +4.3 |
| NS2 | 2.553 | 2.157 | +18.3 | 2.49 | 2.24 | +11.1 | 2.227 | 2.20 | +1.2 |
| θ2 | 2.668 | 2.197 | +21.4 | 2.58 | 2.27 | +13.6 | 2.343 | 2.26 | +3.7 |
| EW2 | 3.12 | 2.677 | +16.5 | 3.08 | 2.68 | +14.9 | 2.79 | 2.63 | +6 |
| Mean error | | | 11.1 | | | 8.8 | | | 0.78 |
| Standard deviation of error | | | 8.7 | | | 6.1 | | | 4.5 |

Model 1, which includes no partitions, provides the best agreement with the corresponding frequencies obtained from the monitoring data. For all models, the higher modes agree worst, with errors increasing as more partitions are added. This is because the corresponding experimental second-order modes (NS2, EW2 and θ2) were seen to be relatively invariant to partitioning arrangements over the monitoring period.

3 GPa partition modulus provided a poor match to observations with mean error 24 % for Model 3 and 14 % for Model 2 distributed among all modes. The much better match obtained using the lower modulus is possibly due to the weak cementing of the blocks to each other, the floor and the ceiling.

Apart from the partition stiffness reduction, the effect of global changes in modulus of major structural members was examined. Increasing column modulus by 50 % increased the frequencies evenly by 4.8 %, increasing core modulus by 50 % increased frequencies by an average 10 % for modes 1–3 and 14 % for modes 4–6 while increasing slab and beam moduli resulted in even increases of 3 and 0.8 %, respectively.

Other effects were considered, for example:

- Adjusting mesh with four times finer mesh resolution in each dimension increased frequencies overall by 2 % but the model was unfeasible to run.
- Modelling openings in the partitions, e.g. doors, etc. resulted in reductions up to 3.5 % for 2.5 m high openings and up to 7 % for full-height (3.6 m) openings, but with wide variations among the modes and inconclusive result.
- Modelling the partition to ceiling connection with 0.03 GPa (i.e. negligible) modulus material resulted in mean reductions of 4.8 % for the first set of three modes and 2.9 % for the second set of three modes, offering a possible suggestion for the relatively small changes in the higher modes observed in the prototype.

The overall conclusion is that the 1 GPa partition modulus provides the best match for the first set of three modes and the observed invariance of higher modes cannot be explained away even if the partition to ceiling joint connection stiffness is drastically reduced.

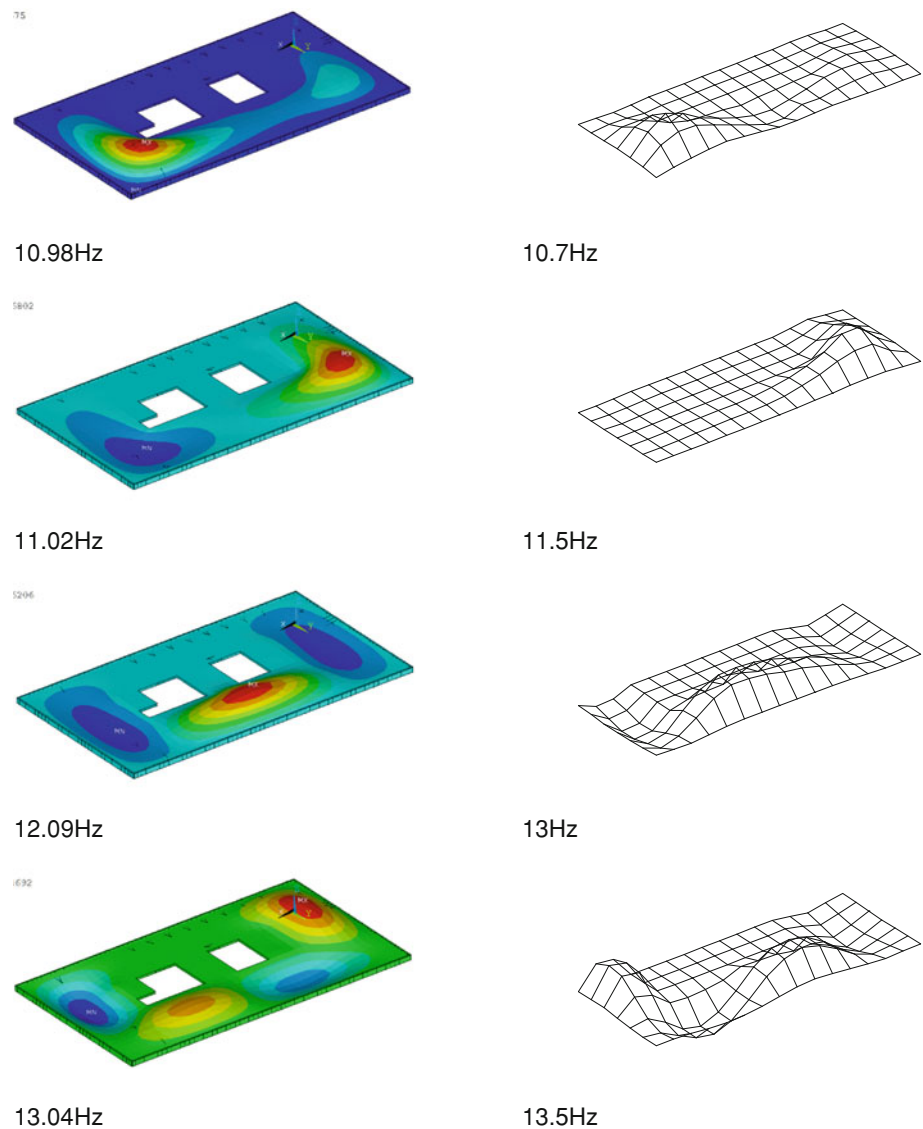### 5.2 Model 4 characterising floor modal properties

The effect of 'non-structural' partitions is more graphically illustrated for floor vertical vibration modes where the reference case is Model 4 representing level 8. Figure 18 shows the model with partitions; in all variants (with and without partitions), columns and core are fixed one level above and below.

Figure 19 compares Model 4 with no partitions with L8_2011 floor mode shapes. The agreement is acceptable; except for the first mode, the model underestimates the frequencies by an average 6 %. The first two modes observed in L8_2011 measurements (10.7 and 11.5 Hz) are relatively weak in Fig. 14 as they involve motion of the slab at the ends of the long bays with little motion in the area of the shaker. Higher modes are stronger and the agreement is more convincing. This all points to Model 4 being able to represent the behaviour of the bare slab reasonably, hence, providing a good basis for studying the effect of partitions.
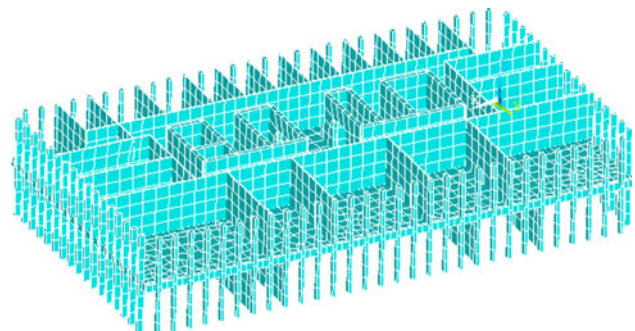


**Fig. 18** Finite element model for floor vibration at level 8

**Fig. 19** Matching of Model 4 (without partitions) with L8_2011 measured modes. FEM modes on *left*, experimental modes on *right*



10.98Hz                                                10.7Hz

11.02Hz                                                11.5Hz

12.09Hz                                                13Hz

13.04Hz                                                13.5Hz

For comparison with L8_2009 measurements, partitions with low Young's modulus were included in Model 4. As for the sway behaviour, using 3 GPa resulted in overestimated mode frequencies, so comparison is given for the 1 GPa variant in Fig. 20. The significant difference from Fig. 19 is that the modes are localised, fortunately in the area where L8_2009 measurements were made, with no sign of the progression of modes along the long bay shown for L8_2011 measurements of Fig. 19.

## 6 Discussion

As a result of the retrofit and removal of masonry partitions, the natural frequencies of the Arts Tower have reduced considerably, by around 10 % for the fundamental modes of each type. This is in addition to the reductions

between the 1987 BRE measurements and the 2009 start of the monitoring.

The main factor in these changes appears to have been the removal of the rigid masonry partitions. Whereas the original masonry partitions contributed significantly to sway stiffness, the lightweight and removable new partitions have had no observable effect. Likewise, removal of the full-height masonry partitions allowed the development of a sequence of plate-like modes of increasing order observed experimentally, with no apparent effect of fixed stud/glass or movable partitions.

Matching observed changes in sway mode frequencies required a relatively low elastic modulus for the partition material compared to assumed values for monolithic masonry, reflecting the contribution of the cement joints. Strangely, while similar proportional changes are predicted by FEM, the experimental evidence is that changes in

**Fig. 20** Matching of Model 4 (with partitions) with L8_2009 measured modes



16.8Hz                        15.8Hz

21.1Hz                        20.1Hz

21.4                          21.1

23.1                          23.8
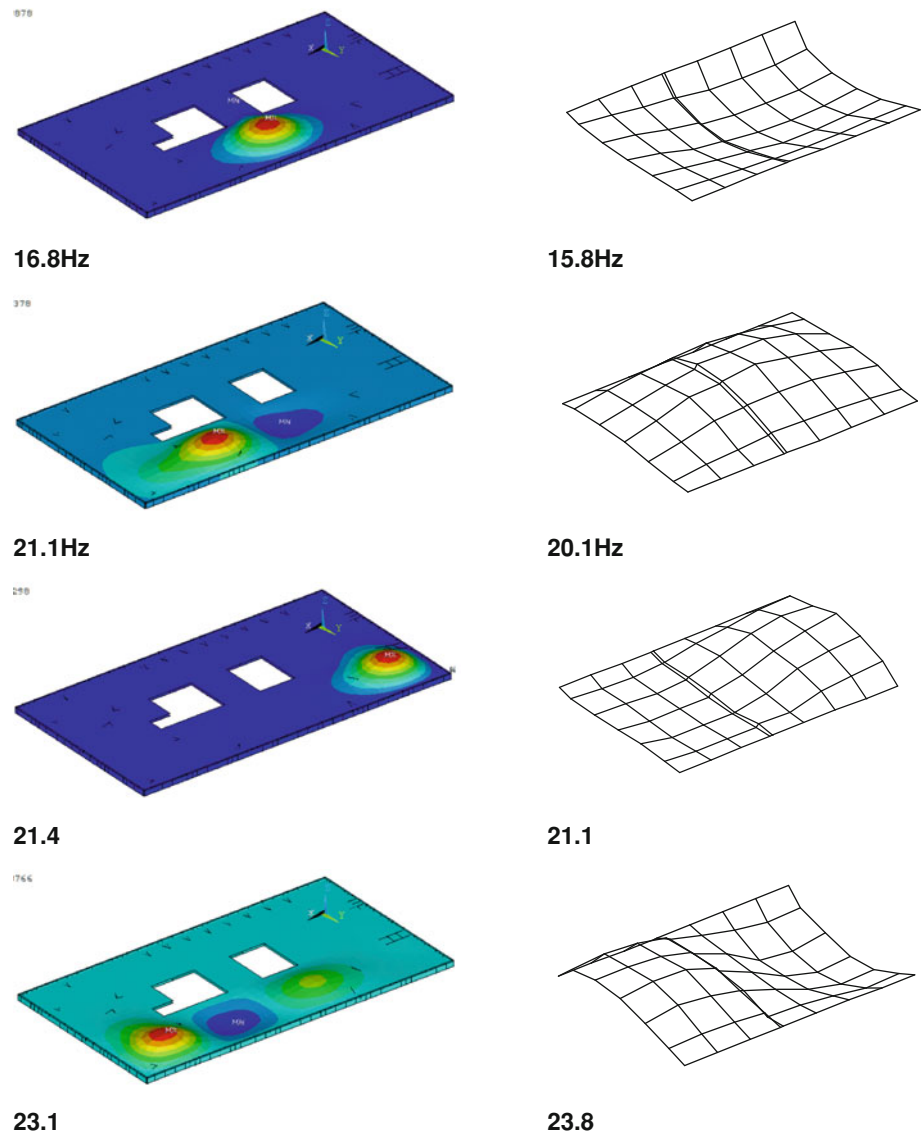
higher mode frequencies have been smaller. This remains a mystery, but since the fundamental modes are most important (in the case of this building) for wind effects, tracking down the modelling error is relatively unimportant.

Before and after the upgrade, the building has also exhibited strong amplitude-dependence of both natural frequency and damping ratio, but there is no clear evidence that the response levels have increased despite the reduced stiffness.

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