

Chapter 6

Structural Health Monitoring of Sheffield University Arts Tower During Retrofit

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Abstract The 78 m Arts Tower at the University of Sheffield has been refurbished during the period 2009–2011, with improvements that included replacement of facades and partitions. The structural changes have been reflected in changes in dynamic properties that have been tracked by a pair of roof-top mounted tri-axial seismometers. The response data have been analysed in real time and accessed by web-viewer, capable to show that natural frequencies have changed dramatically during and after the retrofit. The monitoring system has demonstrated significant variations in modal parameters at different response levels.

The information from the exercise has been used to validate performance predictions by the consulting engineers.

Keywords Retrofit monitoring • Changes in dynamic properties • Frequency drop • Stochastic sub-space identification • Performance prediction

6.1 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 completed in 1965 and is now a Grade II listed property, recognising its heritage status and imposing strict requirements on structural alterations.

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The building has 21 storeys above ground floor level, 2 basement levels, and is approximately 80 m high, 36 m wide and 20 m deep. 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. There are two frame arrangements; above the first floor there are 94 perimeter columns at 1.2 m centres, all rectangular cross-sections of 203 mm by 406 mm; below the first floor there are 16 perimeter square columns of 965 mm side. A transfer slab exists at level 1. Each floor is sub-divided by non-load bearing 75 mm thick ‘breezeblock’ masonry partitions. The basement houses lecture theatres and plant rooms which extend outside of the footprint of the main tower. First floor is 8.54 m above ground level and there is a mezzanine level in between. Normal floors are spaced at 3.55 m intervals.

Structural lateral stability is principally provided by the two reinforced concrete shear cores acting as vertical cantilevers. Lateral load is transmitted to the core 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 claimed they clearly perceived movements of the upper floors and this prompted a vibration study by Building Research Establishment (BRE), published in 1978 (Littler 1988). The study concluded that the significant contribution of the non-structural components such as partitions, walls, cladding to lateral stiffness have prevented an unsatisfactory performance in wind. It was also noted 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, which remained until partitions were removed during the retrofit.

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.

The plan was that in Phase 1 the external façade/curtain walling would be replaced from 1st floor to the crown and lifts (including an unusual paternoster lift) would be refurbished. Floors 9–19, badly needed by the University’s Architecture Department were to be refurbished first. Then in Phase 2, floors 1–8 would be refurbished. Refurbishing a floor would entail removing all partitions and installing a mixture of fixed and movable lightweight partitions.

Figure 6.1 shows the building at various stages of the operation; with original façade and following its replacement, while Fig. 6.2 shows the internal floor plan before and after the retrofit and Fig. 6.3 shows the original and replacement partitions. The heavy masonry partitions, arranged as shown in the original floor plan, have been largely demolished in the lower left view in Fig. 6.3 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 no partitions, others have a combination of fixed and sliding partitions, as shown.



Fig. 6.1 Arts tower during (*left*) and after (*right*) retrofit

Although vibration levels have apparently remained within recognised tolerance limits for occupants, the removal and subsequent replacement of the partitions led to concerns about vibration performance, that have been addressed by analytical studies by the consultant and subcontracted long-term vibration measurements. Both aimed to identify the contribution of the non-structural elements to the building performance.

In order to assess and understand the vibration performance a combination of vibration measurements finite element (FE) analyses and investigations have been made, organised by the Consultant (Gifford), the Vibration Engineering Section (VES) at the University of Sheffield and Full Scale Dynamics Ltd, a spin-off of the University of Sheffield.

Sample tests during the retrofit showed that cube strength varies from 28.8 MPa to 70.5 MPa in the main floors, an average value of 50 was assumed in the analyses, leading to a dynamic modulus of 36 GPa.

6.2 Vibration Studies

Two modal (vibration) surveys of the building have been conducted, one in 1987 lasting 9 days and using artificial excitation and one in 2011 lasting an afternoon and using ambient vibration.

Additionally two long term monitoring exercises have been carried out spanning the complete retrofit period.

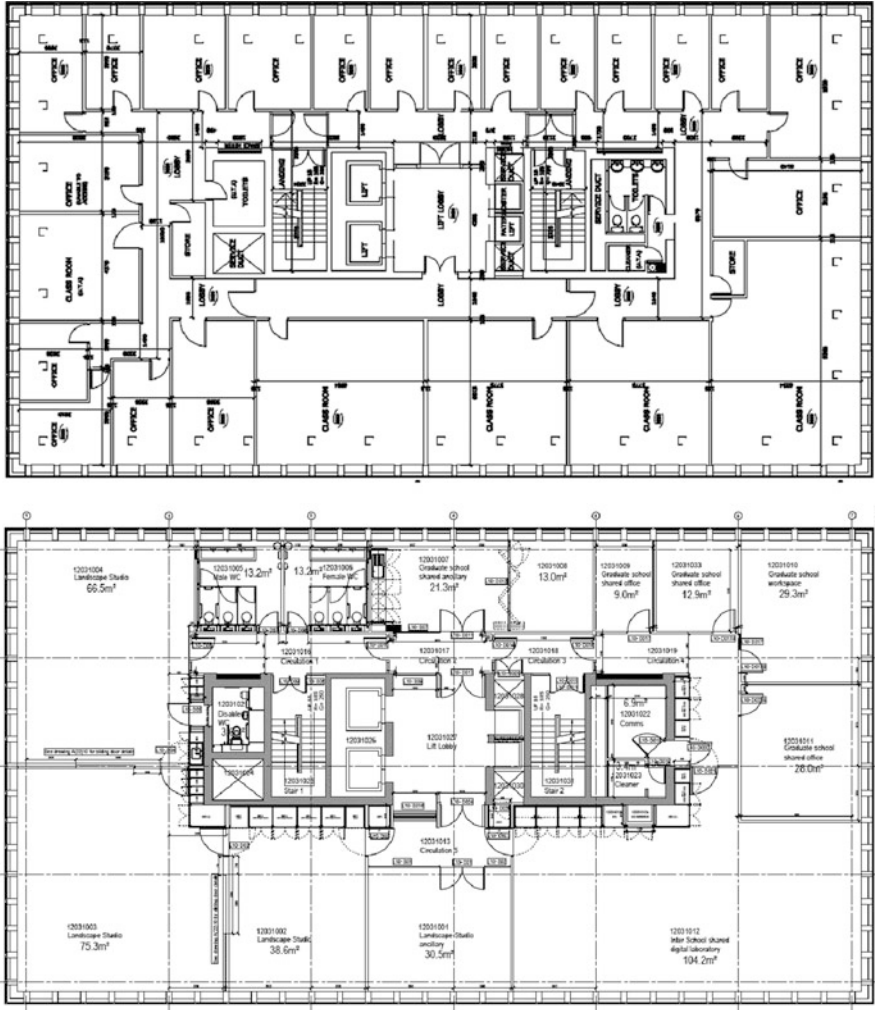


Fig. 6.2 10th floor plan showing two cores and partitioned office spaces before and after retrofit



Fig. 6.3 Demolished masonry partitions, cleared floor and new movable partitions

6.3 BRE Forced Vibration Test, 1987

The first vibration study of the building was a 9-day campaign in July 1987 by Building Research Establishment (BRE)(Littler 1988). BRE used 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 (limited by bearing capacity), sufficient to excite building response in single modes at vibration levels exceeding those likely to be experienced in strong winds. Step sine approach was used, incrementing excitation frequency at 0.1 Hz interval from 0.1 to 10 Hz. The force characteristic of the REM shakers is not uniform due to the ω^2 characteristic for a given eccentric mass and varying armature rotation speed ω ; the mass needs to be reduced in stages as speed increases to keep within the bearing capacity. Natural frequencies and damping ratios were estimated by curve-fitting to the modulus of transfer function which is the ratio of response amplitude to exciting force, the latter being computed based on shaker armature speed rather than measured directly and not containing phase information. More accurate frequency and damping estimates were obtained from the free decay of single mode 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 around the building. Measurements were made at all 23 levels from upper roof to lower ground.

REM shakers are now relatively uncommon for testing civil structures due to severe logistical constraints, a notable contemporary example being the NEES facility at UCLA (Stewart et al. 2005). Because heavy equipment is not required, the ambient vibration survey (AVS), coupled with operational modal analysis (OMA) is now the standard procedure, as used for subsequent studies on the Arts Tower. Shaker testing (with more portable broadband electro-dynamic shakers) is still used for testing smaller structures of components such as footbridges and floors.

Modal parameters for identified modes are summarised in Table 6.1, for the largest shaker force. Values obtained from 20% of full force that would be associated with modest wind speeds are also given, and occur at higher frequencies. This table also reports results from subsequent exercises, for comparison. The long

Table 6.1 Observed mode frequencies

Mode:	NS1	01(TH1)	EW1	NS2	02 (TH2)	EW2	03 (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)	0.626	0.752	0.806	2.157	2.197	2.677	–	–	–
8/2010 (low)	0.610	0.720	0.770	2.240	2.270	2.680	–	–	–
11/10 (high)	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	4.93

Low = weak ambient vibration, high = with strong winds or shaking, ≥ 10 mm/s² RMS

(36 m) axis of the building is the more closely oriented to the East–West direction, hence the lateral sway in the direction of the short (20) axis is labelled '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 labelled θ (TH).

6.4 VES Ambient Vibration Survey, June 2011

An AVS of the building was carried out over 3 h on the afternoon of 10 June 2011, using 24 accelerometers (QA750). On two reference levels (floors 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 remaining 18 accelerometers were arranged at six other levels for a single recording of 16 min; shows the arrangement.

The 18 accelerometers were then moved to different floor levels for another 16 min recording. A planned third recording was scrubbed due to a 5 PM deadline to end the testing. Subsequent OMA using the eigen-system realisation algorithm (ERA) implemented in MODAL software (Brownjohn et al. 2001; Brownjohn 2003) identified the 2D characteristics of modes up to 15 Hz. A selection of mode shapes is presented in Fig. 6.4. While the first six modes can be readily compared with the BRE counterparts, the available description of the BRE mode

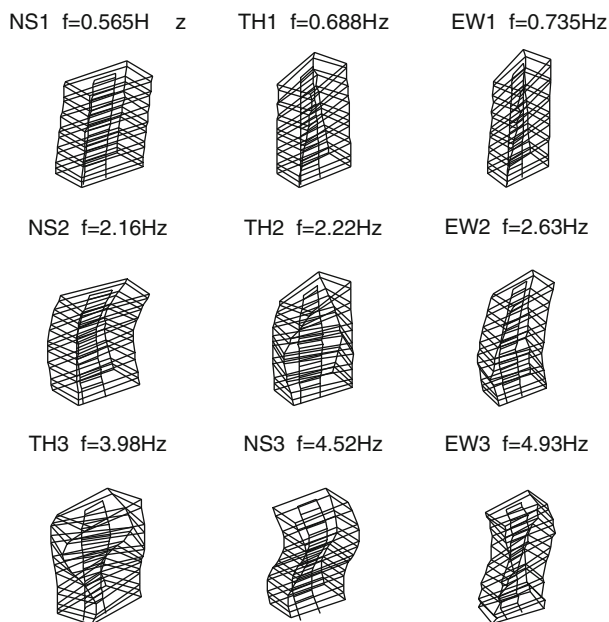


Fig. 6.4 Mode shapes from ambient vibration test, June 2011

shapes does not permit reliable comparison for higher modes. Clearly the mode frequencies have dropped significantly, which reflects the recent structural alterations as well as any degradations taking place over the intervening years.

6.5 Vibration Monitoring Exercises

To investigate these changes in more detail, and to correlate with the analytical studies, vibration monitoring of the building has been organised in two stages:

- By Strainstall, on behalf of Gifford for University of Sheffield Estates Department.
- By Full Scale Dynamics Ltd for University of Sheffield Estates Department.

These exercises are described below.

6.6 Vibration Monitoring by Strainstall UK Ltd

Four accelerometers and an anemometer were installed on and sensing in the direction of the four sides of the building. A logger was triggered on strong wind events to record short time histories. Analysis of data from this system the strongest event during this monitoring in January 2009, provided modal frequency estimates reported in Table 6.1. Wind data are not available, but the largest 1-second broadband RMS response reached 23 mm/s².

6.7 Vibration Monitoring by Full Scale Dynamics Ltd

In September 2009, during the refurbishment of the University of Sheffield Arts Tower, a vibration monitoring system was installed by Full Scale Dynamics Ltd (FSDL). The system, shown in Fig. 6.5, comprises a pair of Guralp CGM-5TD triaxial accelerometers mounted at two locations on the roof and synchronised by GPS. Six channels of acceleration signals are digitised and transmitted to a server for storage and processing.

In Fig. 6.5, the local axes of the two sensors are designated N and E, corresponding to the fixed internal labelling system used for this type of instrumentation. However the major (long) axis of the building is aligned close to the geographic East–West axis and hence the convention used by BRE identifying East–West (EW) for movement along the long axis and North–South (NS) for movement transverse to the long axis. The twisting or torsional vibration modes designated θ are also labelled TH.

Power spectral densities (PSDs, in square root form) of NS and EW response for 4 January 2011, a breezy day with winds between 4 and 8 m/s are shown in Fig. 6.6



Fig. 6.5 Triaxial accelerometers and GPS antennae for FSDL system

for displacement (top), velocity (middle) and acceleration (bottom). The response levels are an order of magnitude smaller than for the strongest winds, but the data show features relevant for discussion.

In the PSDs mode NS1 (0.56 Hz) only appears in the NS response, while modes NS2 and 02 (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. Higher modes dominate the acceleration.

Displacements and accelerations are derived by integration or differentiation of the seismometer velocity signals, which are shown in Fig. 6.7 as a spectrogram, for the NS component. The 0–3 Hz bandwidth is shown, covering the lowest six modes, which can be distinguished by close inspection of the bands.

The raw time series for the NS component are shown in Fig. 6.8 for broadband and in Fig. 6.9 for the 0–1 Hz band containing the first three modes. Displacements (row 1) are very small and will be dominated by the first three modes. The contribution of fundamental modes to the total acceleration is clear from comparison of row 3 in the two figures and inspection of Fig. 6.6.

The strongest response levels recorded during the monitoring, reached 1.2 mg RMS (12 mm/s² RMS) over a 30 min period, still rather less than the response levels achieved by the Strainstall monitoring. The BRE forced vibration exercise produced 12.3 mm/s² RMS for a single mode, equivalent to 0.6 mm harmonic amplitude.

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

Automated analysis on the server includes calculation of RMS acceleration and estimation of modal parameters (frequency, damping and mode shape ordinate)

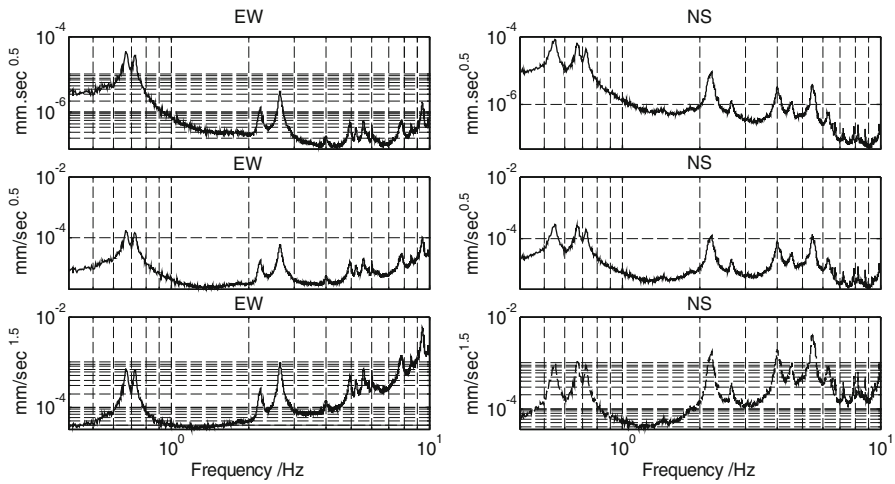


Fig. 6.6 Broadband displacement power spectra for 4 January 2011

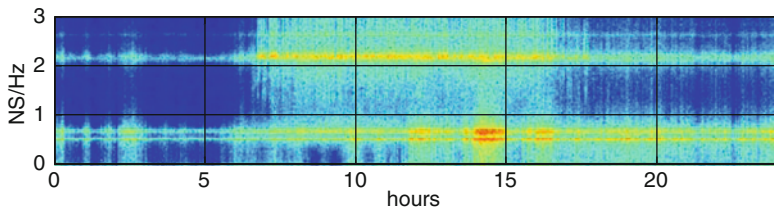


Fig. 6.7 Spectrogram of NS velocity corresponding to Fig. 6.6

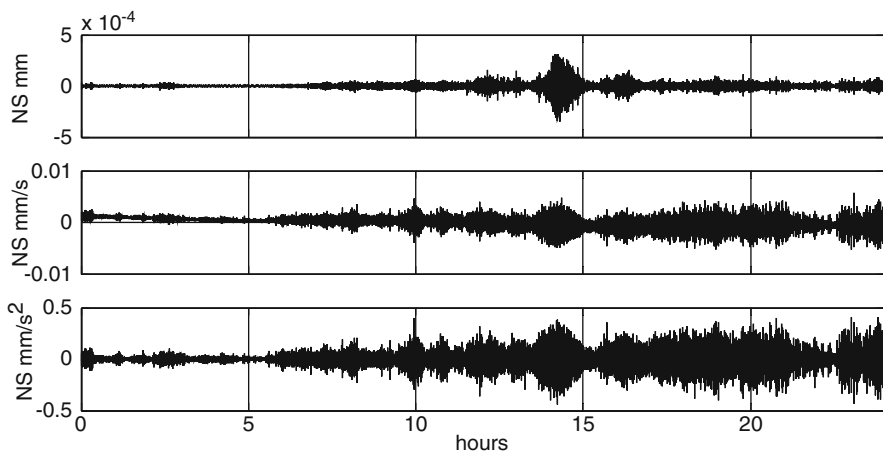


Fig. 6.8 Arts tower motion in NS direction on 4 January 2011

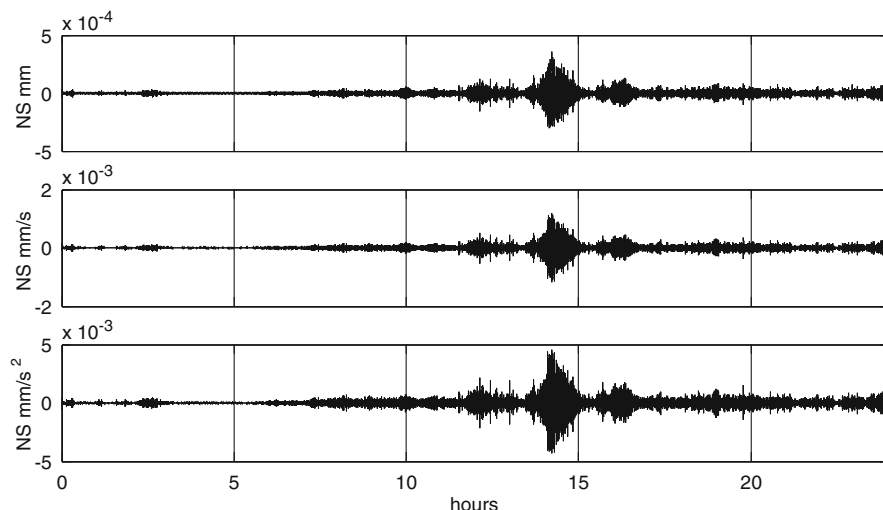


Fig. 6.9 Arts tower motion in NS direction on 4 January 2011, 0–1 Hz components

based on the previous 30 min of data, using the stochastic subspace identification (SSI) technique (Peeters and De Roeck 1999). A web viewer operating in Google Apps environment and a MATLAB interface can be used for accessing the raw and post-processed data. The web viewer has been used for generating the performance plots used in this paper, for example Fig. 6.10, which shows the environmental loads and first mode frequencies for the duration of the monitoring.

Variations of temperature (between -10 and 30°C) and wind speed (up to 15 m/s or 33.5 mph average) would be reflected in some way in the modal properties (as discussed later), but these cannot account for the most remarkable features which are the two periods of significant frequency reductions at the start of the monitoring (September and October 2009) and then September and October 2010.

6.8 Construction Events

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

Demolition of the original façade was carried out from September 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

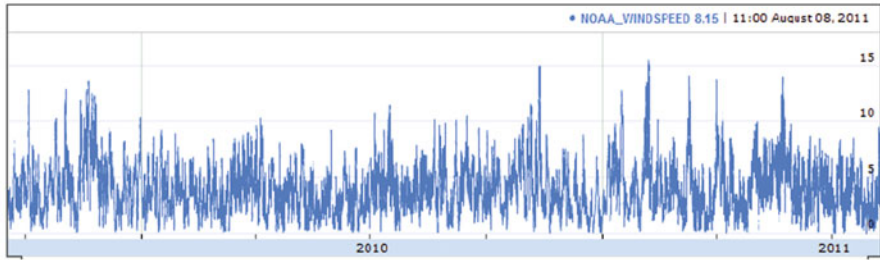
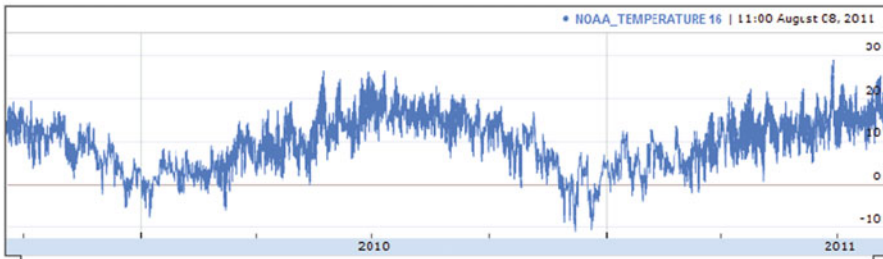
Wind Speed (m/sec)**Temperature (°C)****Frequencies (Hz)**

Fig. 6.10 Meteorological conditions and fundamental mode frequencies for duration of FSDL system. Data span 17 September 2009–8 August 2011

cannot be seen, however the rise in frequencies over mid-summer 2010, even with 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 one week per floor. This period corresponds to the fastest and most significant drop in frequencies.

Higher order modes exhibit changes during the construction period but to a much smaller extent than for the fundamental modes.

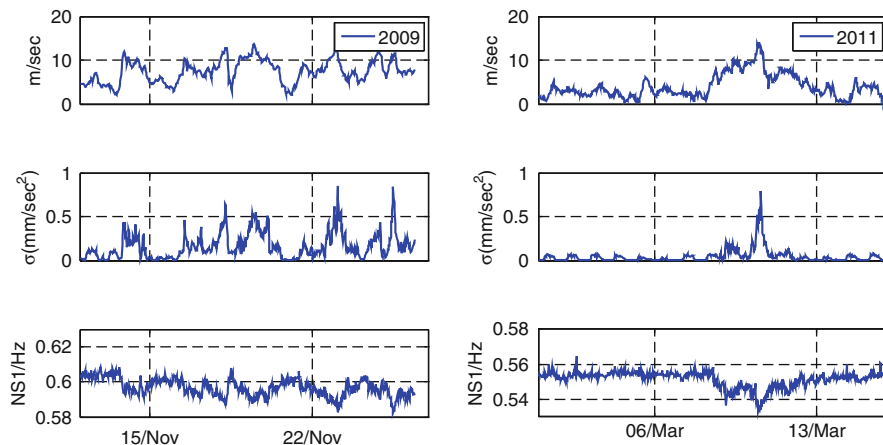


Fig. 6.11 Large amplitude response examples before and after retrofit: wind speed, RMS acceleration and mode frequency

6.9 Amplitude Effects

For two periods when frequencies were not changing due to construction events, the relationship of response amplitude with frequencies is very clear. Figure 6.11 shows for both periods clear daily RMS variations.

For both periods, frequency drops are clearly linked to RMS response level, with similar amplitude dependence in higher order modes. Note the RMS is broadband, which is dominated by the lower frequency modes for strong winds.

The relationships before and after the retrofit are presented in Fig. 6.12, with no obvious difference between the two periods for all the relationships. The lack of obvious difference of amplitude dependence on wind allays fears of the effect of reduced stiffness. Where narrow band RMS values (focused around specific modes) are available the trends are similar and not clarified.

6.10 Damping Changes

Figure 6.13 shows damping for mode EW1 throughout the monitoring period and zoomed on the period around February 4, 2011.

There are large gaps of no reliable damping data, but data appear to show slight increases in damping after the retrofit. What is clearer is that damping is increased for higher amplitude response. It is well known that damping values derived from this type of OMA are subject to considerable variance error (Reynders et al. 2008)

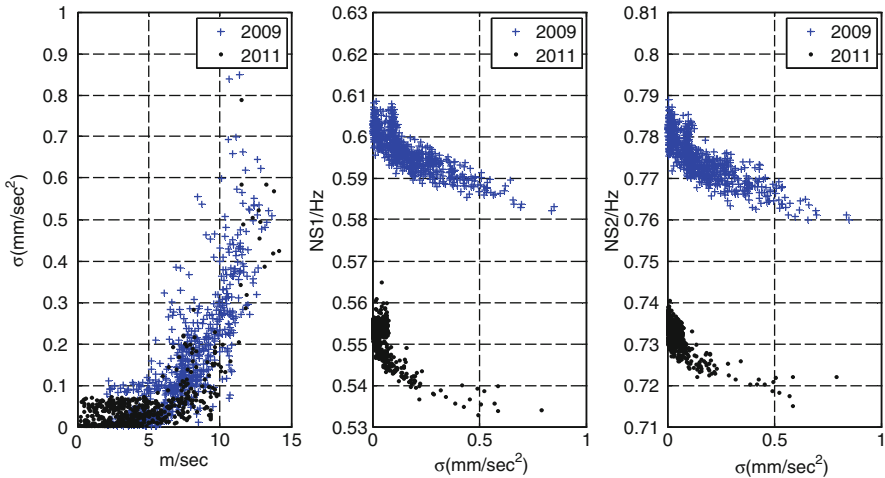


Fig. 6.12 Wind-amplitude and frequency-amplitude dependencies before and after retrofit

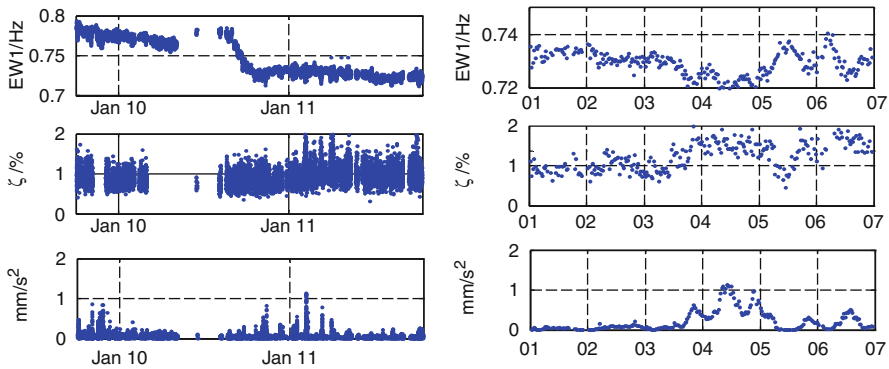


Fig. 6.13 Damping and frequency of mode EW1

and that the free decay method applied by BRE (one benefit of expensive shaker testing) is the most accurate procedure so these results should be regarded as qualitative rather than quantitative.

6.11 The Further Mystery of Second Mode Frequency Changes

Figure 6.14 shows trends in the lowest four translational modes as percentages of the mean value for the duration. The data points plotted correspond to the lowest levels of response, below 0.2 mm/s².

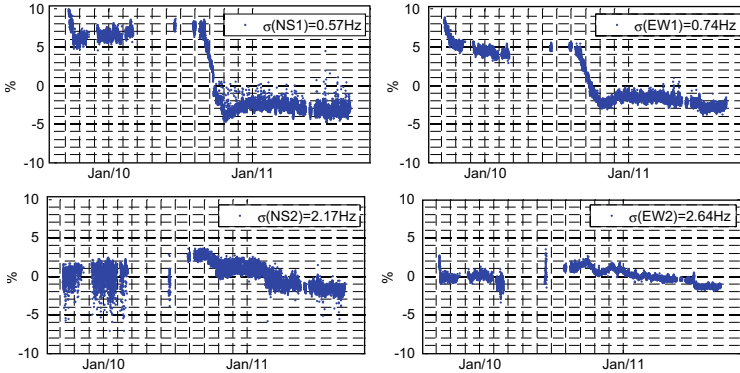


Fig. 6.14 Percentage changes in first four translation modes

The two horizontal modes (NS2 and EW2) display a rather different pattern to NS1 and EW1. First, the changes are proportionally smaller. In particular the dramatic reduction in NS1 and EW1 frequencies is replaced by a modest rise followed by a modest fall, reaching a minimum frequency simultaneously with NS1 and EW1 frequencies. From then until the most recent measurements, NS2 and EW2 frequencies drop steadily, and more significantly (even in % terms) than NS1 and EW1. Ranges of EW1 and EW2 are larger than for NS1 and NS2.

6.12 Structural Causes of Frequency Changes

According to the measurements, the strongest influence on the dynamic properties has been the internal rearrangement of the partitions. Hence the effects of partitions on dynamic performance of the building were studied by finite element simulations.

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 (Su et al. 2005; Pan et al. 2006) and an office tower (Brownjohn et al. 2004). All these studies point to the stiffening effect of non-structural elements that plays a role in the low-level vibration response.

6.13 Finite Element Modeling

To study this effect a finite element model (FEM) of the Arts Tower was created using ANSYS software. BEAM44 element was used for modeling columns and beams (perimeter and transfer), with SHELL63 used for slabs, core walls

Table 6.2 Matching of measured mode frequencies and analytical estimates

Mode	Frequency (Hz) with all partition			Frequency (Hz) with partitions at levels 1–9			Frequency (Hz) no partitions		
	FEM	EMA 21/09/2009	% Error	FEM	EMA 20/10/2009	% Error	FEM	EMA 31/10/2010	% Error
	NS1	0.647	0.626	+3.3	0.609	0.61	0	0.523	0.54
EW1	0.758	0.752	+0.7	0.737	0.72	+2.3	0.636	0.67	-5
TH1	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	2.12	+1.05
EW2	2.668	2.197	+21.4	2.58	2.27	+13.6	2.343	2.19	+6.9
TH2	3.12	2.677	+16.5	3.08	2.68	+14.9	2.79	2.63	+6

and partitions. This model was developed in three variants simulating the building at various stages of the retrofit:

Model 1: representing the final state of the Arts Tower after removing partitions and including main structural elements (columns, slabs and core walls).

Model 2: represents the state of the Arts Tower after the first partition removal phase (removal of partitions at levels 9–19)

Model 3: representing the original building before the retrofit i.e. with all partitions in place.

Several assumptions were made in the modeling. First linear elastic material was used for the analyses, with uniform concrete modulus based on cube strength of 28.5 MPa.

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

Third, 0.1 m thick partitions were assumed to be fixed at the slabs with 1,400 kg/m³ density and 3 GPa Young's Modulus. Moreover, the element types used for the modeling are shown in Table 6.2.

Fourth, despite some variation of partition arrangement from floor to floor, partitions for each floor were modeled as they exist on floor 1, but accounting for any major differences.

6.14 Modal Analysis

Modal analyses were done for each variant. While mode shapes did not vary noticeably among the variants, natural frequencies changed did vary, particularly for the first three modes (NS1/EW1/TH1).

Using the assumed Young's modulus for the partitions led to poor frequency matching. The value 3 GPa represents homogeneous material, whereas breeze

blocks in the partition are cemented together. The partition failure mode in the demolition clearly indicated that the cement joints were relatively weak, and when the overall Young's modulus was reduced to 1 GPa, an acceptable match was obtained with experimental observations. This matching is shown in Table 6.2.

Comparison is made against measurements at the lowest response level, consistent with the fully linear assumption of the FEM. The EMA estimates for 9/2009 are with partitions removed from levels 19 down to 12 (we did not catch the original state when monitoring started) so the estimates are lower than would be for low level excitation with all partitions present i.e. before the retrofit.

6.15 Alternate Finite Element Modeling and Performance Predictions

BAM Construction, responsible for the retrofit, contracted Gifford to investigate the condition of the Arts Tower and also to develop necessary repair strategies. As part of the investigation Gifford developed FE models of the building before and after the retrofit. Table 6.3 shows that the Gifford model (which was developed before the retrofit) overestimates the frequency drop compared to our model (which was adjusted to match known final frequencies).

The Gifford FEM was used to predict performance of the building before and after the retrofit. In particular, using the prevailing wind loading code, acceleration response levels were predicted to increase by 33% with the removal of the partitions, resulting in a 'response factor' of $R = 8$ during a 1 in 50 year storm. A response factor $R = 1$ represents the threshold of perception for acceleration, and $R = 8$ is acceptable for a busy working environment. On the basis of these predictions the Arts Tower performance seen to be acceptable after the retrofit and requiring no structural measures such as bracing to limit response.

Moreover Fig. 6.12 shows no obvious change in relationship between wind conditions and response. Changes in mass distribution due to all the alterations are not known (although removing masonry partitions is likely to have reduced mass) but the frequency drops will be primarily due to stiffness reduction. The only means of compensating for reduced stiffness will be increased damping, but the evidence for that is not clear.

Table 6.3 Gifford and Sheffield FEM modal frequency estimates

Mode	Frequency (Hz) with all partitions		Frequency (Hz) with no partitions	
	FEM (UoS)	FEM (Gifford)	FEM (UoS)	FEM (Gifford)
NS1	0.65	0.64	0.52	0.47
EW1	0.76	0.70	0.64	0.60
TH1	0.86	0.80	0.75	0.66

6.16 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 reductions between the 1987 BRE measurements and the start of the monitoring.

The main factor in these changes appears to have been 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. Although it is the subject of a separate investigation, only the masonry partitions had any effect on the slab vertical (bending) vibration modes. Matching observed changes in sway mode frequencies required a relatively low elastic modulus for the partition material. Strangely, while similar proportional changes are predicted by FEM, the experimental evidence is that changes in higher mode frequencies have been smaller.

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 dependences of frequency and damping on response amplitude or of response amplitude on wind speed have changed.

6.17 Epilogue

Building monitoring exercises are not always straightforward, particularly in the presence of ongoing construction activities.

Building on the success of the dual seismometer monitoring system, an upgraded monitoring system was planned and an biaxial accelerometers installed on the two core walls at four levels. During subsequent retrofit of one of these levels, four clearly labelled accelerometers were destroyed.

A Digitexx Corporation PDAQ Structural Health Monitoring System was being installed in the Arts Tower basement when asbestos contamination was discovered in the basement plant room. The area was sealed off for a year (until 9/2011), and all plant room machinery was destroyed along with all power and internet connections. The PDAQ survived due to its hermetic seal and is now in use.

The roof-mounted Guralps survived the retrofit and have continued operation past the contracted monitoring period thanks to the attention of Estates office Staff and Sheffield researchers.

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