Paper

Dynamic testing and numerical modelling of the Cardington Steel Framed Building from construction to completion

B. R. Ellis, BSc, PhD, CEng, MIStructE
Building Research Establishment

T. Ji, BSc, MSc, PhD, CEng, MIStructE
University of Manchester Instituite of Science & Technology
(formerly Building Research Establishment)

Synopsis

This paper is concerned with the dynamic behaviour of an eight-storey steel-framed building. The building construction was undertaken in discrete stages, five of which are described. At each stage the natural frequencies of the building were measured using a laser system. A comprehensive forced vibration test was also performed when the building was complete to determine all of the characteristics of the fundamental modes of vibration. Both types of test are described, and the results that were obtained are presented. In parallel with the dynamic tests, numerical modelling has been conducted at each stage. The finite element model is described, and the main calculated results are presented. The measurements and calculations are then compared to provide a better understanding of the building behaviour and to identify where the numerical model needs to be improved. Finally, the benefits from the combined experimental and numerical studies are discussed.

Introduction

In many branches of engineering, feedback from the performance of prototype structures is used to refine and improve designs and is considered an essential part of development procedures. However, in structural engineering, where structures are often designed on an individual basis and where the testing of complete structures is rarely undertaken, there is a lack of feedback on measured performance, except perhaps for the rare cases when failure occurs. Thus an important mechanism for improving designs and understanding structural behaviour has been neglected.

BRE has been involved in dynamic testing for many years and has tested many different structures including several tall buildings[1]. The question which arises is whether feedback from such tests can be used to improve analytical or numerical models, and this will be examined in this paper. There are many factors which serve to complicate the modelling of buildings, like the influence of floors, walls and cladding, and it is here where the Cardington building provides a unique opportunity. The building, which is described in detail in the next section, has been constructed in several well defined stages, providing an opportunity to measure the overall characteristics at each stage, and thereby evaluate the influence of the various structural elements.

In parallel with the dynamic testing, numerical modelling of the building has been conducted. The twin purposes of the numerical modelling are to provide additional detailed results which are not available from tests, so that the understanding of the dynamic behaviour of the building can be improved, and to check the accuracy of numerical modelling using experimental results so that the finite element model can be improved. It is perhaps worth emphasising that the numerical modelling required to determine the building's dynamic characteristics is essentially the same as the model which would be required to predict 'static' behaviour or evaluate safety under given loads; hence the results have far wider implications than just for dynamic behaviour.

This study aims to:

- provide information about the dynamic characteristics of the Cardington building at different construction stages
- understand the effect of structural components, such as floors and walls, on the dynamic behaviour of the building
- identify the modelling errors involved through parametric study and by comparison with measurements

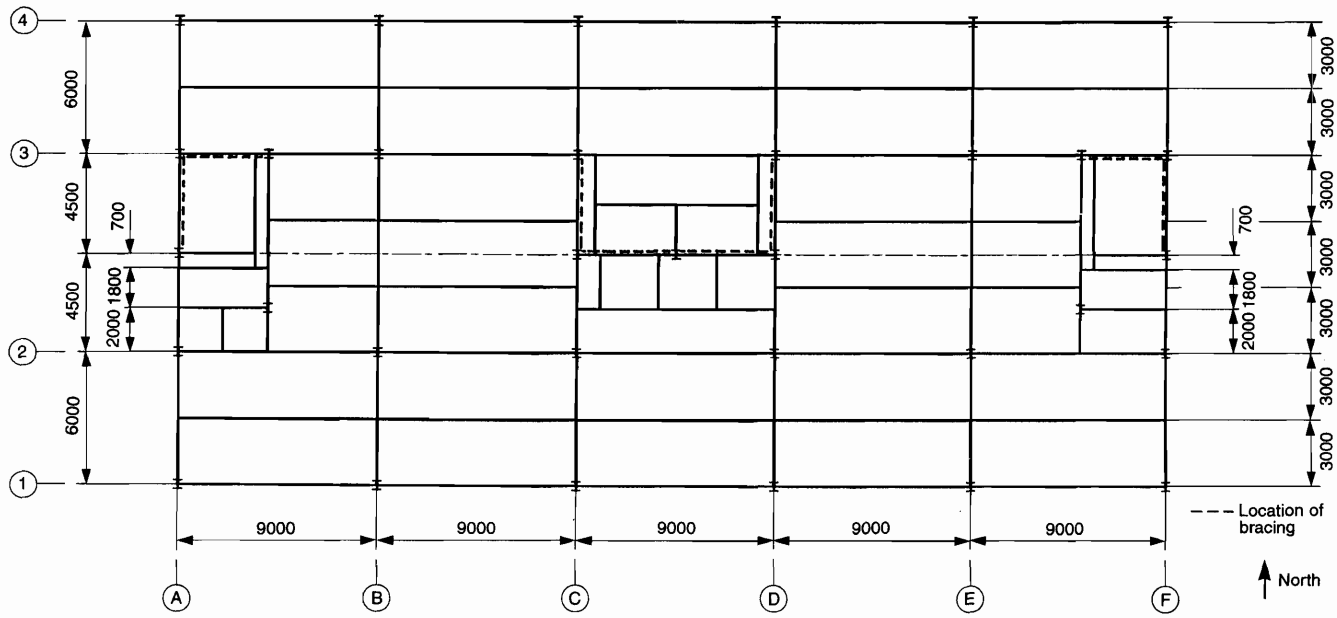


Fig 1. A plan drawing of the building showing the beam and column layout for floors 3 to 7

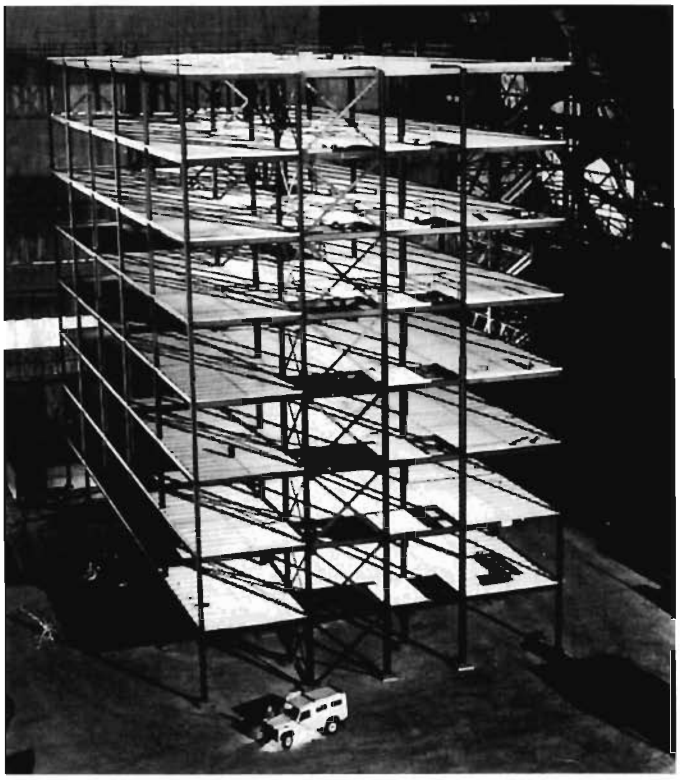


Fig 2. The building at stage 2

The Cardington Steel Framed Building

BRE has been developing a large building test facility at its Cardington laboratory. A large, strong floor was cast in the laboratory to support full-sized buildings. The first structure to be built, which is the subject of this paper, is a steel framed building representing an office block. It has eight storeys with a height of 33.5m, five bays in its 45m length (east - west) and three bays in its 21m width (north - south). The building is designed as a no-sway structure with a central liftshaft, and two staircases that are braced to provide the necessary resistance to lateral construction and wind loads. A typical plan for floor 3 and above, showing the location of the bracing, is given in Fig 1. At the ground and first-floor levels the arrangements are slightly different with an open entrance area at the centre of the south of the building. The main steel frame is designed to transmit vertical loads, and the connections between beams and columns are designed to transmit vertical shear only[2]. The composite floors consist of profiled steel deck and insitu lightweight concrete. Blockwork walls are provided in the end faces to full height and to dado height on the side faces.

The construction has been undertaken in discrete stages, and those which will be considered in this paper are:

stage 1: the bare steel frame
stage 2: the frame plus steel floor decks
stage 3: the frame plus composite floors
stage 4: the frame plus floors and walls
stage 5: the frame plus floors, walls and static loads

For stage 1 the structure consisted of the steel frame including its bracing members before the steel floor decks were added. The steel frame comprised four column cross-sections and five beam cross-sections, depending on location. However, stage 1 is hypothetical, because the floors were assembled as the framework was constructed. Nevertheless, this is an important stage in the numerical modelling.

At stage 2 steel floor decks were assembled at all levels. The building at this stage is shown in Fig 2. The profiled steel decking, which used 0.9mm sheets, was continuous over a minimum of two spans along the north-south direction.

At stage 3 the composite floor system was finished, with the profiled steel decking, shear studs, reinforcing mesh and lightweight concrete. The completed system gives an overall floor depth of 130mm.

For stage 4 the external walls were erected. Blockwork walls were provided in the end faces (east and west ends) to full height and to a dado height along the side faces (900mm above floor levels).

For the final stage of construction, static loads were applied to the building to represent live loads. 208 sand bags were placed on each floor, except the uppermost floor where 169 bags were used and the fifth floor which was empty. The 1100kg sandbags were almost uniformly distributed on the floors. Fig 3 shows the building at this stage.

Dynamic tests

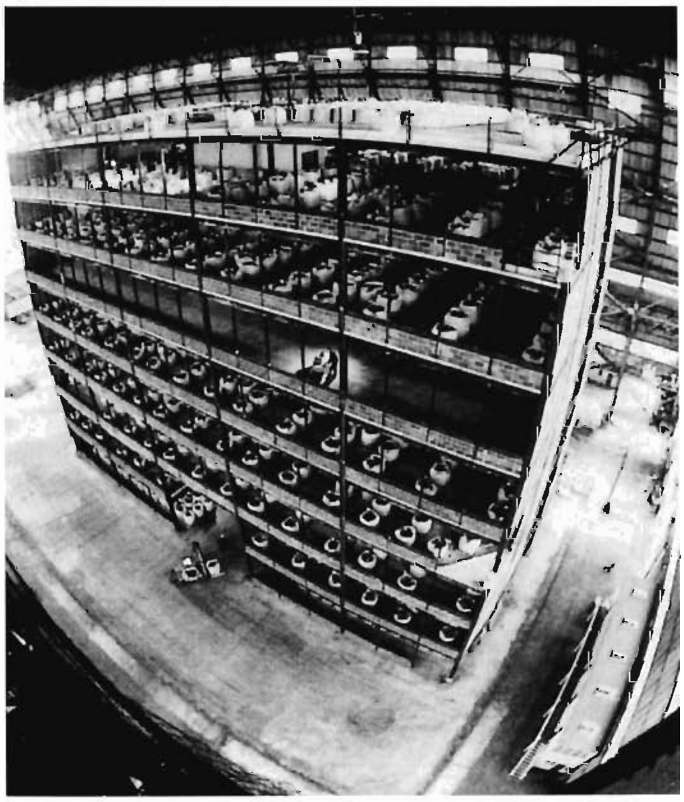
The main objective of the tests on the Cardington building was to determine the characteristics of the fundamental modes of vibration. Two types of test have been undertaken. The first makes use of laser measurements to monitor the ambient response of the structure, i.e. its natural vibration caused by air movement within the laboratory. These measurements are processed to identify the frequencies of the modes of vibration. The laser has the advantage that the measurements can be made remotely, and no equipment needs to be placed on the structure. This is useful for taking measurements during construction or after any incident which causes damage. The second type of test is the forced vibration test which is used to determine all of the characteristics of the fundamental modes of vibration, i.e. frequency, mode shape, stiffness and damping. This provides detailed information but requires the use of specialist test equipment attached to the building. The low amplitude measurements taken primarily using the laser system are used for comparison with calculated values.

Laser tests

The key part of the system is a long-range laser interferometer developed for measuring vibrations in-line with the laser beam. The main output is an analogue voltage which is proportional to velocity. The basic measuring procedure involved aiming the laser at specific positions on the building. The signal from the laser was fed into signal conditioning equipment and the output recorded on a computer for 409.6s at a sampling rate of 200Hz. The velocity-time history was then processed using a fast Fourier transform technique to calculate an autospectrum which shows the frequency content of the signal. Fig 4 shows the autospectrum of one record at stage 5.

The system was set up at two positions with the north-south (NS) position being restricted by the side of the laboratory. For the east-west (EW) measurements the laser was situated 110m to the west of the building and for the NS measurements it was situated 35m from the south of the building. Five recordings were taken on each of the west and south faces for analysis, the five positions being the two top corners, the top centre and a corner column and the centre position on the fifth floor. Although it is not possible to define the different modes from one spectrum, the five measurements help to clarify the situation, i.e. measurements in the centre of the

Fig 3. The building at stage 5



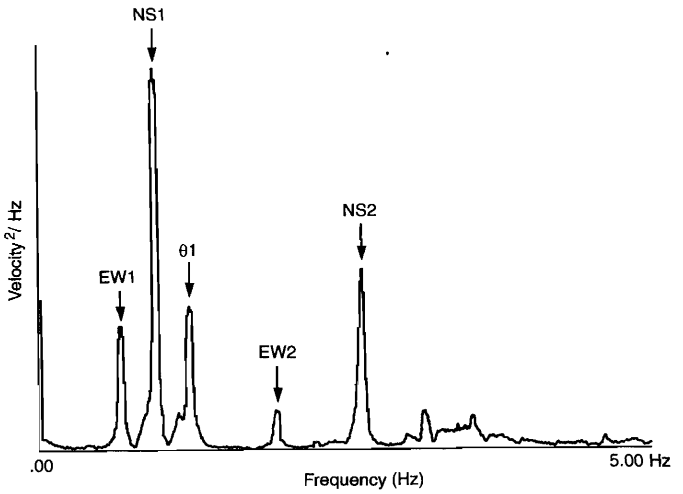


Fig 4. An autospectrum measured at stage 5

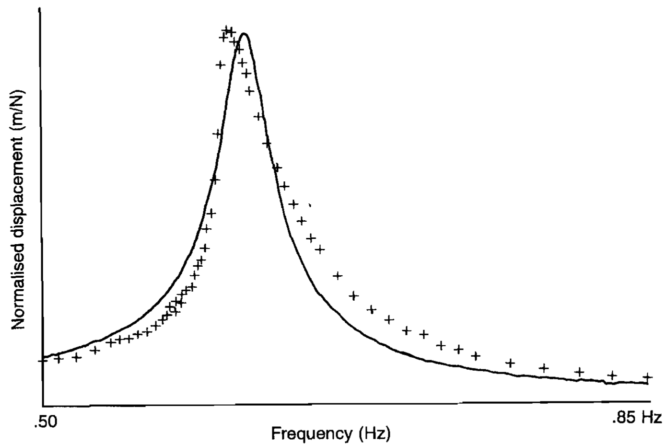


Fig 5. EW1 response spectrum at stage 5

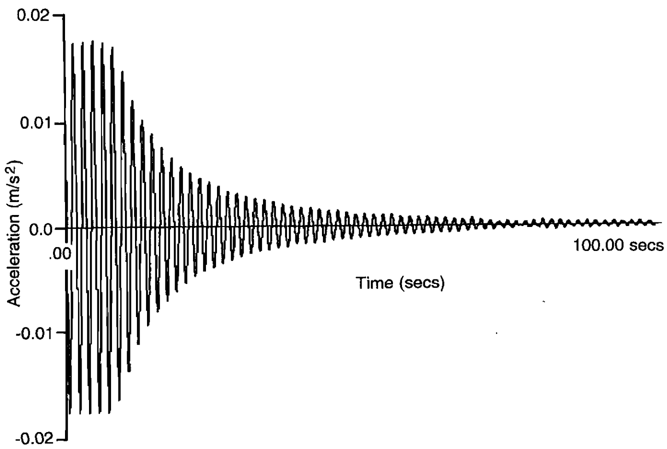


Fig 6. Decay of vibration for EW1 at stage 5

south face would tend to eliminate the EW and torsion modes, and measurements at two-thirds the height of the building would tend to eliminate the second-order modes. A summary of the results is given in Table 1.

Forced vibration tests

To measure all of the characteristics of the fundamental modes, a comprehensive test set-up is required. This involved the use of vibration generators to shake the structure in a controlled manner. The vibration generators used for the tests have a pair of contra-rotating masses which provide a unidirectional sinusoidal force. There are four generators which are controlled from one computer. The system works within the frequency range 0.3 to 20Hz, with the resolution being 0.001Hz.

The building was tested in April 1995 when it was complete, i.e. at stage 5. The exciters were placed in the four corners of the top storey in the orientation for exciting the mode to be investigated. The exciters were set to produce a known force and then the forcing frequency was incremented over the required frequency range. The response was monitored using an accelerometer aligned to measure motion in the appropriate direction. The response was sampled by computer using optimised filtering, amplification and curve fitting. The response was then normalised by converting the measured accelerations to the equivalent displacement and then dividing this displacement by the applied force. A best-fit curve based on a viscoelastic model was determined for the resulting spectrum. The parameters used to determine the best-fit model for each mode are frequency, damping and stiffness, and are assumed to have constant values.

The response spectrum obtained for the tests in the EW direction is shown in Fig 5. The crosses on the figure represent the measured values, and the continuous line represents the best-fit one degree-of-freedom curve. It can be seen that the fit is not perfect and the measured values have a characteristic negative skew compared to the best-fit curve. This is typical of this type of measurement and shows one aspect of non-linear behaviour which has also been observed in other structures[1]. The values obtained from this test are given in Table 2.

Mode shapes were determined by setting the exciters to provide a steady-state motion at a natural frequency and monitoring the response at various locations throughout the structure relative to a reference accelerometer. To obtain further measurements of damping a decay of vibration was recorded for each mode. This involved shaking the building at its natural frequency and recording the response when the excitation was suddenly stopped. The decay of vibration for the fundamental EW mode is shown in Fig 6.

Non-linearities observed in vibration decays

If the results of both the laser and the forced vibration tests are examined for stage 5, significant differences can be seen between the frequencies, even though they were obtained on the same day. These differences are a function of the non-linearities in the structure which can be shown by analysing the measured vibration decays.

To determine values of frequency and damping a best-fit decay based on

TABLE 1 – Measured frequencies at different construction stages

Stage	EW1	NS1	θ1	EW2	NS2
1a*	0.98	1.22	1.71	3.30	3.42
2**	1.31	1.55	1.67		
3	0.69	0.83	0.89	2.10	2.44
4	0.75	1.31	1.64	2.13	3.81
5	0.66	0.93	1.22	1.90	2.63

* tested when the basic framework was erected plus the lower four steel decks
** tested using a small vibration generator at amplitudes comparable to the laser tests, the damping being 0.53% for EW1, 0.45% for NS1 and 0.84% for θ1.

TABLE 2 – Results of forced vibration test in stage 5

Mode	Frequency (Hz)	Damping % critical
EW1	0.617	2.25
NS1	0.804	2.46
θ1	0.959	3.28
EW2	1.835	2.79

TABLE 3 – Frequency and damping evaluated for various sections of a decay

Relative amplitude	Natural frequency (Hz)	Damping % crit.
1.000	0.611	2.87
0.366	0.636	1.81
0.181	0.645	1.28
0.106	0.647	1.02
0.062	0.656	0.85

a visco-elastic model is derived for a selected part of the measured decay, using a least squares fitting procedure. The fitting procedure varies frequency, damping and initial amplitude. Five contiguous 10s samples were selected from the measured decay (Fig 6) and viscoelastic decays fitted to the measured data. The tail of the measured decay is disregarded as this is affected by the ambient excitation of the structure. The data extracted from the five samples are given in Table 3 and related to the amplitude of vibration at the start of the first sample. Fig 7 shows the measured and best-fit curves for the first 10s sample. From the table it can be seen that the change in frequency covers the variation observed between the results of the laser and forced vibration tests.

Numerical evaluation of the dynamic characteristics

The building has been modelled and analysed using the LUSAS general purpose finite element program[3]. The modelling is based on the engineering design and site observations. To allow the 3-dimensional function of the frame, a 3-dimensional thick beam element (BMS3) is used to model the beams and columns. The function of the bracing members is to transmit tension or compression, therefore they were modelled as 3-dimensional bar elements (BRS2). The floors and walls were modelled in two ways using either 3-dimensional thin shell elements (QSI4) or space membrane elements (SMI4). A coarse mesh is adopted to model the global dynamic behaviour of a structure. To maintain consistency in the analysis at the different stages, the mesh division and number of nodes is constant, but a different number of elements is used. The basic information about the finite element modelling and the building is summarised in Table 4, and includes the building weights at the construction stages. The results of the calculations for the six lowest frequency modes at each of the five stages are shown in Table 5. A plot of the FE model at stage 1 is shown in Fig 8.

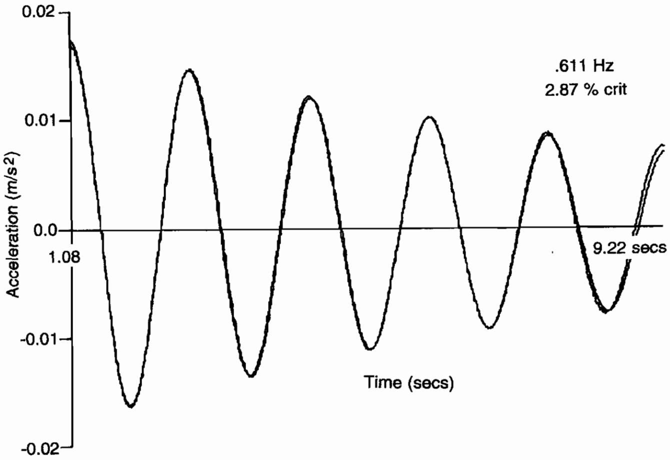


Fig 7. Measured and best-fit decays for part of the decay shown in Fig 6

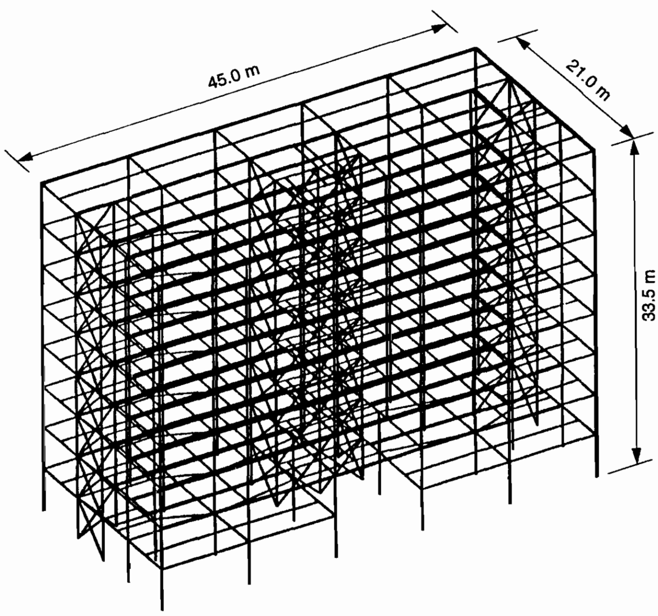


Fig 8. FE model of the building at stage 1

TABLE 4 – Summary of FE models at different construction stages

Stage order	No. of BMS3	No. of BRS2	No. of QSI4	No. of SMI4	Total elements	Total notes	Building weight (t)
1	1374	128	0	0	1502	764	325
2	1374	128	318	0	1820	764	373
3	1374	128	318	0	1820	764	2302
4	1374	128	318	126	1946	764	2612
5	1374	128	318	126	1946	764	4170

TABLE 5 – Calculated frequencies and mode characteristics at different construction stages

Mode order	1	2	3	4	5	6
Frame (1)	1.03	1.13	1.14	1.18	1.83	1.91
	L θ	EW B L	NS L	L	L2	L2
Steel decking (2)	1.60	1.67	1.91	3.15	3.34	3.38
	EW θ	NS θ	θ EW	EW2	TI	BI
Concrete floors (3)	0.75	0.77	0.89	2.54	2.66	3.15
	EW θ	NS	θ EW	EW2	NS2	θ2
Walls (4)	0.89	1.95	2.78	2.83	4.39	4.49
	EW	NS	θ	EW2 θ	F	F
Loading (5)	0.70	1.55	2.22	2.27	3.37	3.38
	EW	NS	EW2 θ	θ	F	F

The characteristics of the modes are described using abbreviations as follows:
L local vibration θ rotation TI twisting in the floor plane
EW east-west direction NS north-south direction
BI bending in the floor plane F floor vibration
2 second order deflection over the height of the building

Some of the main features of the calculated structural characteristics are:

Stage 1 At stage 1 the structure consists of the pure steel frame with the bracing arranged locally rather than globally (Fig 1). This creates an uneven distribution of lateral stiffness in the structure. As a result, the modes show a combination of global and local vibrations, and the frequencies of the first few modes are close. Eight different frame models have been analysed to examine the effects of various element arrangements, mesh sizes, element cross-sections, and joint restraints.

Stage 2 At this stage the steel decks were assembled at all levels. The profiles of the decking sheet in the two main directions are different, and the different cross-sections are represented using the same cross-section with orthotropic material properties. Both membrane and shell elements were evaluated for modelling the decking. The membrane model gave the lower frequencies, and it is the results of this model which are given in Table 5. The frequencies of the decked frame are significantly higher than those of the pure frame structure, because the decks provide effective stiffness in each floor plane while adding relatively little weight to the structure. Due to the steel decking, the local vibrations which showed in the pure frame structure disappear. Examining the first three modes, it seems that, when the steel decks are erected, the structure acts as a whole and the rotation couples with the EW motion but not the NS movement. This is because the lateral stiffness of the structure is dominated by the bracing and the arrangement of the bracing for the frame is symmetric for the east and west halves but not for the north and south halves of the structure.

Stage 3 The composite floor was modelled using a mass/unit area of 340kg. This is higher than the design drawings specify and follows a site study on the construction loads for the composite floor slabs in the building[4], where it was found that the slab thickness is up to 30mm greater than design specification and the concrete density was approximately 8% higher than that expected. To consider the different cross-section properties of the floor in its two principal directions, an orthotropic material model is adopted. The floor provides a large stiffness in the floor plane and strongly connects all columns; however, it provides little lateral stiffness to the building but adds significant weight and consequently the structural frequencies decrease. The shapes of the first three modes are similar to those when shell elements are used for modelling the steel decking. If the concrete

TABLE 6 – Comparison of calculated and estimated frequencies

Mode order	1	2	3	4	5	6
Steel decks f_s	1.76	1.80	2.15	5.93	6.40	7.14
Composite floors f_c	0.75	0.77	0.89	2.54	2.66	3.15
Estimation (eqn.(1))	0.71	0.72	0.86	2.39	2.58	2.87

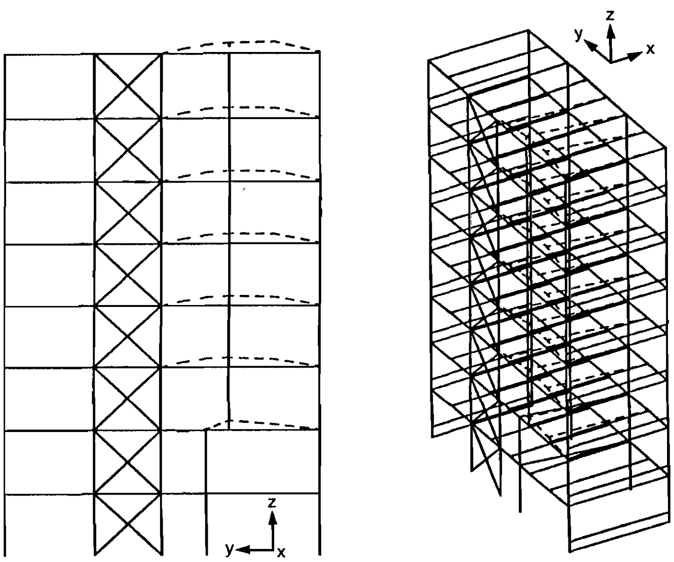
floors do not contribute stiffness to the building and the structural mass is almost uniformly distributed over the height of the building, the frequencies obtained from these two stages should have the following approximate relationship:

$$f_c = f_s \sqrt{\frac{M_s}{M_c}} \qquad ....(1)$$

where f_s, f_c, M_s and M_c are the frequencies and total masses at stages 2 and 3. Table 6 gives the comparison of the calculated frequencies at the two construction stages and the ones estimated using eqn.(1) based on the stage 2 frequencies calculated using the shell elements. The estimated values are lower than the calculated ones because the stiffness contribution from composite floors is neglected in eqn.(1). The comparison of the values given in the last two rows in Table 6 provides an indication of the stiffness contribution from the composite floors.

Stage 4 Both membrane and shell elements were evaluated for modelling the walls. As expected the membrane model predicts slightly lower frequencies. The walls contribute significant stiffness to the building which is reflected in the mode shapes and frequencies. The walls at full height at the east and west ends increase the frequency from 0.72Hz to 1.95Hz in the NS direction. The walls on the north and south sides which are less than a quarter of storey height increase the EW frequency from 0.71Hz to 0.89Hz. The first two modes show a uniform movement in the EW and NS directions due to the symmetric arrangement of the added walls that provide a dominant component of stiffness to the building. The walls were erected at the four sides of the building and therefore the structural ability to resist rotation is significantly increased. A structural weakness is identified at this stage due to the columns at the inner side of the entrance area being offset by 2m from the columns on the upper floors. This leads to a relatively low frequency mode of vibration of the floors from the second storey upwards, and hence a mechanism to transmit vibration from one floor to another is formed which results in a potential serviceability problem. Fig 9 shows the mode shape, with the left drawing showing the shape on a sectional elevation and the right drawing providing a 3-dimensional view of a section of the building between axes D and E.

Stage 5 After the above four stages of construction, the structural parts of the building were basically completed. The building is now loaded using sandbags which are almost uniformly distributed on the floors. Therefore, the added mass results in reduced frequencies with little change to the mode shapes.



Sectional elevation along D axis

View of a section of the building

Fig 9. Mode shape for the floor vibration

TABLE 7– Relationships of frequencies between stage 4 and 5

Mode	EW1		NS1		θ1		EW2	
	Measured	Calculated	M	C	M	C	M	C
Unloaded building (ω)	0.75	0.89	1.31	1.95	1.64	2.78	2.13	2.83
Loaded building (ϖ)	0.66	0.71	0.93	1.57	1.22	2.24	1.90	2.31
ω^2/ϖ^2	1.29	1.55	1.98	1.55	1.80	1.54	1.25	1.51

Comparison between calculation and measurement

To improve the understanding of the structural behaviour and to examine possible modelling errors it is useful to compare the calculation and measurement; however, because no experimental data are available at stage 1 the accuracy of modelling the bare frame with its assumed boundary and joint conditions cannot be assessed.

At the second stage where steel decking has been erected, the FE model overestimates the measured frequencies of the first two modes by 22% and 8% and shows that the modes have a combination of translational and torsional motion which could not be deduced from the frequency measurements but could be anticipated from the asymmetry in bracing. Because a comparison was not undertaken at the previous stage, it is not possible to isolate the errors due to modelling the decking from those due to the model of the overall framework. The effect of the different elements on modelling the decking has been studied and it is found that the membrane elements yield lower frequencies than shell elements, but in both cases the calculated frequencies are bigger than the measured values.

When the composite floors are cast at stage 3, modelling the floors is straight forward in spite of the orthotropic material properties which are used for the substitution of the different cross-sections in the two principal axes. The calculated and measured frequencies are basically in agreement, although slight differences are to be expected. The comparison indicates that the FE model should be less stiff in the EW direction and more stiff in the NS direction, and that the FE model shows a combined movement of EW translation and torsion in the fundamental mode which could not be determined from the laser measurements. The calculated values for the second-order modes show larger differences than for the fundamental modes.

At stage 4 when the walls were erected, the increase in the calculated frequencies is bigger than the increase in measured values. Also the increase in the NS and torsion modes is far larger than for the EW modes. This shows that the walls in the calculation are significantly stiffer than they should be, especially for the full height walls on the east and west faces. This provides clear and useful feedback for identifying errors in the model of the walls. The walls at the two ends are actually divided into several areas by beams and columns and in the construction of the walls, gaps between the walls and columns and beams are unavoidable. However, in the numerical modelling it is difficult to represent the real situation accurately and the assumption of continuity between walls leads to significant differences between measurements and calculations.

Comparing the measurements at stage 4 and stage 5 in which the only difference is the loading the added mass should reduce the frequencies proportionally; in other words, the ratios of the first few frequencies at these two stages would be very close as observed in the numerical analysis. Table 7 provides a comparison between frequencies at the last two stages.

The calculated frequencies basically obey the relationship of eqn.(1). However, the measured ones show differences, i.e. the first and fourth ratios are almost the same and about 20% lower than the numerical mass ratio, while the second and third ratios are about 20% bigger than the numerical mass ratios. This comparison suggests that owing to the loading, the structural stiffness in the NS direction decreases while that in the EW direction increases.

At this stage the results from the forced vibration tests are also available. Fig 10 shows both measured and calculated mode shapes, which for presentation purposes have been evaluated close to the eastern stairs. It can be seen that there are consistent differences between the measured and calculated mode shapes. This provides further information about the stiffness distribution in the building which is useful for improving the numerical model.

When setting up the equipment for the forced vibration tests, significant vibrations were noticed on the top floor produced by movements of a loaded trolley on the fifth floor over the entrance area. This corresponds to the floor vibration mode which was identified at stage 4 of the numerical model and suggested to be a potential problem in the structure.

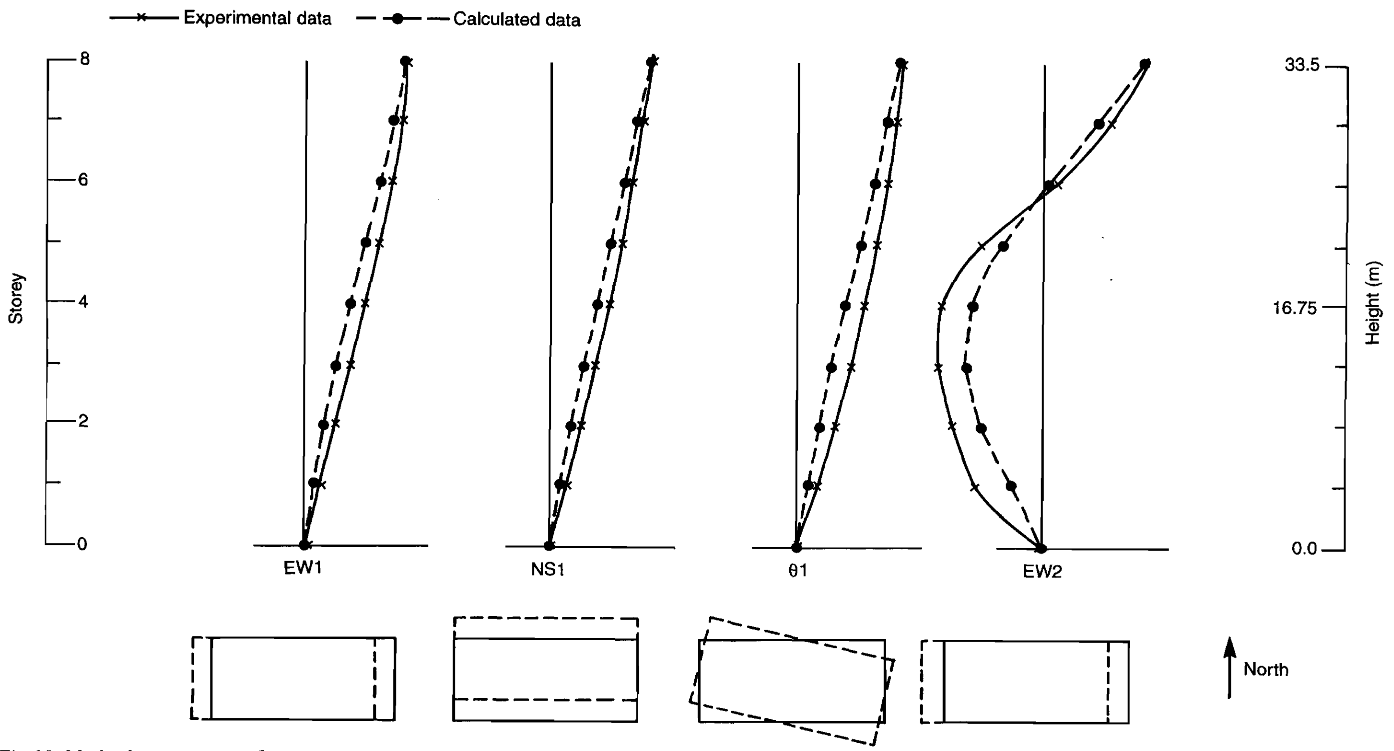


Fig 10. Mode shapes at stage 5

The benefit from the combination of experimental and numerical studies

In this study two types of experimental measurement have been used. The laser tests were quick and cheap and provided the structural frequencies, while the forced vibration tests were far more comprehensive and provided detail of frequencies, mode shapes, and damping. With both types of measurement, the information can be considered to be accurate but incomplete (i.e. only a few modes are examined). On the other hand, the numerical study gives complete but inaccurate results. Thus the experiment and numerical studies are complementary, and their combined usage is sensible.

One way of using the experimental results would have been to refine the numerical model at each construction stage; however, the model presented has not been 'improved' using the experimental data, but instead it provides an example of both typical and accumulated errors.

The experimental measurements by themselves provide some useful results. For example the measurements at stages 4 and 5 showed that the loads affected the structural stiffness. This was not expected and may provide the basis for improving the modelling of the loaded building.

The numerical modelling was also instructive and identified a floor vibration problem, due to the architectural requirement that some columns around the entrance area were offset from the columns at higher levels. This suggests where further experimental tests would be beneficial. This vibration problem could be cured by providing infill walls between the first and second floors and between the central column and the edge column along the C and D axes.

However, the combination of experimental measurements and numerical modelling provides further information. To give one example, consider the major source of inaccuracy in the numerical model, which is modelling the infill walls. The infill walls have a dominant effect on the overall stiffness of the structure and are therefore important for any framed structure. The tests shown here provide a clear indication of the stiffness contribution from both full height and dado height infills, both of which are significantly lower than the increase predicted by the numerical model. This provides an opportunity to refine the numerical model and also to evaluate various methods for modelling infills which have been proposed worldwide.

This study helps to achieve a better understanding of one particular building, although by itself it does not provide general guidance for engineers. A similar type of study was undertaken considering the performance of concrete large panel structures[5], using dynamic measurements taken by BRE on finished buildings, and it is by repeating such studies and learning from them that advances will be made. For engineers, the individual benefit will be achieved by providing some feedback from their own structures tested when they are complete and learning about the accuracy of their design models. For the engineering profession, the benefit would accrue from reports and findings of such work undertaken by individual engineers and the combination of findings from feedback on many different types of structure. The mathematical model of the structure is important as it forms the basis of design and safety evaluations, and the measurements can provide some feedback into the design process. The need for improved accuracy in producing overall building models cannot be overemphasised[6], and this will become more and more important in the future with increasing use of computer models in design.

Concluding remarks

The main aim of the experimental work presented in this paper was to measure the overall structural characteristics which could be used for comparison with the numerical model. The fact that measurements on the Cardington building are made at many different stages of the construction helps to identify the contribution of items like floors and walls to overall stiffness. It is suggested that providing some feedback mechanism, possibly checking design calculations against frequencies measured when a structure is complete, will to a certain extent help to maintain the safety standards in buildings. The conclusions from this study are summarised as follows:

(1) Dynamic testing

The laser tests were simple and quick and were ideal for measuring the frequencies of the building. However, they were limited because no measured mode shapes were obtained. The forced vibration tests were far more comprehensive and took far longer than the laser tests but yielded much more detailed information about the fundamental modes. The combination of both types of measurement of the frequencies identifies the non-linearity of the system.

(2) Numerical modelling

Difficulties are encountered for modelling the steel decking and walls. The modelling of the structure from construction to completion identifies the effect and contribution from the main structural elements, such as floors and walls. Parametric studies help to identify the degree of modelling error. The finite element modelling of the building reveals its dynamic behaviour, some aspects of which can be anticipated from the structural layout of the building, e.g. asymmetry in the building leads to coupled translation and torsion motion in some modes.

(3) Structural dynamic behaviour

At the frame stage, the lateral connections between structural elements are not strong and the structural stiffness is unevenly distributed. Therefore, local and global vibrations are coupled and the first few frequencies are close. At the steel decking and concrete flooring stages, the floor provides a large, horizontal stiffness and strongly connects all columns, and the local vibration consequently disappears. However, the floor systems only slightly affect the lateral stiffness of the structure. At these stages, the structure vibrates as a whole, and the lateral stiffness of the structure is symmetric in the east and west halves but not in the north and south halves of the structure (Fig 1). Therefore, the vibration modes present a combined movement in the EW direction and rotation and an almost independent movement in the NS direction. At the last two stages, the walls provide dominant lateral stiffness to the structure, with resulting significantly increased frequencies and altered modes shapes. The first two modes clearly show a uniform movement in the EW and NS directions respectively. The structural stiffness changes as the building is loaded. A structural weakness is identified in that the vibration on any floor between the second floor and the roof can be transmitted to other floors over the entrance area. This is because the columns are offset at the second-floor level.

(4) Comparison between calculation and measurement
The best agreement between results from calculation and measurement is achieved at stage 3 when the composite floors are cast. This is because the assumed errors in the previous stages have been reduced, i.e. the effects of the joints and the modelling of the profiled steel decking. The measurements determine the actual frequencies of the building which are useful for identifying possible modelling errors in the numerical modelling and modifying existing models. For instance, by comparing the experimental and numerical results at stage 4, it is recognised that the models of the walls are too stiff due to idealisation errors. A prediction that vibration can be transmitted between floors over the entrance area has been confirmed by observation.

The combined theoretical and experimental study on the structure is important, because an experimental study provides accurate but incomplete information, while a theoretical study supplies complete but inaccurate results.

References

1. Ellis, B. R.: 'Full-scale measurements of the dynamic characteristics of tall buildings in the UK', accepted for publication in a special edition of *Journal for Wind Engineering and Industrial Aerodynamics on Damping in buildings,* 1996
2. Armer, G. S. T., and Moore, D. B.: 'Full-scale testing on complete multi-storey structures.', *The Structural Engineer,* **72,** No. 2, 1994 pp30-31
3. LUSAS: User Manual, Kingston-upon-Thames, FEA Ltd, 1993
4. Gibb, P,. and Currie, D. M.: 'Construction loads for composite floor slabs', *The First Cardington Conference,* November 1994
5. Milne, J. S.: 'Modelling overall building behaviour for design purposes', *PhD thesis,* Heriot-Watt University, February. 1992
6. Ellis, B. R:. 'An assessment of the accuracy of predicting the fundamental natural frequencies of buildings and the implications concerning dynamic analysis of structures', *Proc. ICE, Part 2,* September 1980

Informal study groups

The purpose of the study group scheme is to create opportunities for members of the Institution to exchange ideas and work on deepening and developing their knowledge of structural engineering, thus stimulating a greater interest in and promoting the art and science of structural engineering.

Members wishing to take part in the work of a study group or who require further information about a study group should write to the appropriate Convener.

History of Structural Engineering

Convener: Frank Newby, MA(Cantab), FEng, FIStructE, HonFRIBA,
27 Mayfield Avenue, London W4 1PN
The Structural Engineer, March 1973, p110

Model Analysis as a Design Tool

Convener: F. K. Garas, PhD, CEng, FIStructE, MICE,
Taylor Woodrow Construction Ltd., Taywood House, 345 Ruislip Road, Southhall, Middlesex UB1 2QX
The Structural Engineer, February 1977, p63

Qualitative Analysis of Structural Behaviour

Convener: D. Johnson, BSc(Eng), PhD, CEng, FIStructE, MICE,
Department of Civil & Structural Engineering, Nottingham Trent University, Burton Street, Nottingham NG1 4BU
The Structural Engineer, November 1978, p309

Computing in Structural Engineering

Convener: P. Gardner, BSc(Hons), MBA, CEng, MIStructE, MICE
Southampton Institute of Higher Education, East Park Terrace, Southampton SO9 4WW
The Structural Engineer, March 1987, p83

Advanced Composite Materials and Structures

Convener: Dr C. J. Burgoyne, Emmanuel College, St. Andrews Street, Cambridge CB2 3AP
The Structural Engineer, Part A, June 1987, p221

Management and Maintenance of Bridges

Convener: G. Davison, BSc, CEng, MIStructE,
c/o The Institution of Structural Engineers, 11 Upper Belgrave Street, London SW1X 8BH
Structural news, 23 January 1990, p4

Arch Bridges

Convener: Professor C. Melbourne, BEng, PhD, CEng, MIStructE,
Dept. of Civil & Environmental Engineering, University of Salford, Telford Building, Salford M5 4WT
Structural news, 15 March 1994, p3

Offshore structures

Convener: Mr A. R. Donaldson, Det Norske Verites, Verites House, Harness Circle, Altons, Aberdeen AB1 4LY

Space structures

Convener: Dr G. A. R. Parke, Department of Civil Engineering, University of Surrey, Guildford, Surrey GU2 5XH