INNOVATIVE INSTRUMENTATION TO STUDY THE BEHAVIOUR OF A HIGH-RISE BUILDING

Dr. Tso-Chien PAN[1], Associate Professor
Dr. James M. W. BROWNJOHN[1], Senior Lecturer
[1] School of Civil and Structural Engineering
   Nanyang Technological University
   Nanyang Avenue, Singapore 639798
   Republic of Singapore
   Phone: (+65) 799-5285 Fax: (+65) 791-0676

Abstract

Standing at 280 m, the 66-storey Republic Plaza building is one of the tallest buildings in Singapore. The structural system of the building comprises a reinforced concrete core wall and a structural steel frame. The steel tube columns, filled with concrete, form an external ring, while the horizontal steel frame systems simply supported at the core wall support a composite slab at each floor. A large number of stress and strain gauges were embedded inside the core wall and the concrete filled tube (CFT) columns. At selected floors, strain gauges were mounted in the steel beams. During construction, the trends observed in stress and strain measurements of the core wall and the CFT columns are generally consistent with the increasing dead loads, while the tends in strain data for the floor beams are more complex. An ambient vibration survey (AVS) was conducted after the completion of the structure. From the AVS measurements of dynamic lateral response, natural frequencies and mode shapes for lower lateral and torsional modes have been obtained. Results of the finite element models for the core wall/steel framing system agree reasonably well with the measured translational fundamental frequencies. However, without a high level of refinement, the finite element models cannot reflect the torsional behaviour. There is no evidence that the curtain wall system affects stiffness or damping properties of the structure at low excitation levels.

1. Introduction

Republic Plaza (Figure 1) is Singapore's tallest building, with a total height of 280 m above sea level (limited by local aviation regulations) and 281 m above the basement. It has 67 levels from the first basement (B1) to roof level (66th storey) and is the subject of a continuing investigation in which static and dynamic responses are being monitored.

The scope of the monitoring project has included:
1. periodic measurements of stress and strain in the major structural elements at one level in the building during and after construction
2. periodic measurements of acceleration response during construction, in order to estimate the natural frequencies and damping ratios,
3. a full-scale ambient vibration survey (AVS) of the complete structure to determine the mode shapes associated with the measured natural frequencies and
4. long term monitoring of wind at rooftop as well as vibrations at rooftop and basement levels to determine the exact nature of natural frequencies and damping and dependence on loading conditions.

1

2. Structural system and construction sequence

A full description of the structural system is given elsewhere [1]. The foundation system comprises six inner caissons founded up to 62 m deep and connected by a 5.5 m concrete mat, plus eight exterior caissons founded up to 40m and linked by deep transfer beams; all caissons are 5 m diameter.

Figure 2 shows the floor plan at level 19 which is above the vertical recess visible in Figure 1 and shown dotted in Figure 2, and below the taper visible in Figure 1. The tower comprises a reinforced concrete (RC) central core, peripheral steel columns, a steel floor beam system between core wall and columns and RC slabs inside and outside the core.

The RC core wall itself is octagonal with (at level 19) outside dimensions 21.5 m along the axis indicated by the arrow labelled 'A' and 22.65 m along the axis indicated by the arrow labelled 'B'. The core extends almost the full height of the building and is 600 mm thick from basement level up to level 39 and 400 mm thick between levels 40 and 62. Above level 62 the floor plans are significantly different with an incomplete 300 mm thick core wall.

The perimeter of the building comprises eight large steel columns, 1.219 m diameter at basement level, and eight smaller steel columns, 1.016 m diameter. Smaller diameter columns are used at higher levels (e.g. 1.1716 m and 0.9144 m diameters at level 19), and up to the 49th level the columns are filled with concrete, i.e. concrete filled tubes (CFT).

Up to and including level 62 the octagonal perimeter bounded by the columns lies within a square with dimension 45 m (Figure 2) and has two tapering sections between levels 20 and 27 and between levels 44 and 46. In these sections, the four sides labelled 'taper' progressively retract inwards while the edge between the small beams narrows and the 45m dimension remains constant up to level 62 (Figure 1). The two mechanical equipment floors which are located above the tapering sections and the Executive Club at level 62 have level heights of 7.9 m rather than the standard 3.95m. The perimeter is enclosed by a curtain wall system attached by brackets to the office slab.

Figure 2 also shows the floor beam arrangement at level 19. The core wall and columns are connected by a main radiating framework of long (to large column) and short (to small column) horizontal structural steel beams. The framing system is completed by a circumferential ring of steel beams connecting the CFT columns and miscellaneous beams connecting to the short main beams. The beam-column connections are moment resisting while the beam-core wall connections are simple pinned connections, as shown in Figure 3. At the foundation level B1, the columns are pinned to the caissons or transfer beams by bolted connections. At the two mechanical floors, diagonal steel beams in the vertical plane are employed as outriggers to limit storey drifts.

Within the core, the low rise (LR) lifts reach to level 35, which permits a core wall opening of 2.95 m high by 10.9 m wide, symmetric about axis B from level 38 upwards. High rise (HR) lift shafts are closed from level 3 to 33 and the eight double-deck HR lifts connect at levels 34 and 35 with the seven double-deck LR lifts. Three other shafts are for VIP, fire and service lifts.

Spanning between the core walls and the CFT columns, the 'office slab' of 125 mm to 135 mm thickness is a composite structure supported by the floor beams with steel decking

used as permanent formwork for the mesh-reinforced concrete. The core wall contains a cast-in-situ 150 mm thick RC core slab of constant dimension (20.3 m by 21.45 m) with various openings for lift shafts, stair wells and service ducts.

Substructure construction began in late 1991, and work on the superstructure began in early 1993. Static response instrumentation was first installed in the core wall at level 18 and the first readings were taken at this stage on 30th October 1993 which is taken as reference time (day 0) for time dependent static and dynamic measurements.

The slip-formed core wall rose fastest, followed by the core slab. Progress of the CFT columns lagged the core wall with a slowing down during steelwork erection at the tapering sections. Office slab lagged two or three levels behind the CFT columns. Installation of curtain wall panels started in February 1994 (by day 125). After completion of the structural system in March 1995 (by day 437), completion of the curtain wall took a further 80 days (by day 522)

In addition to the structural mass, water storage tanks were installed at mechanical equipment floors (rooftop and levels 28, 47 and 65) in mid-June 1995 (day 592) totalling 1.5% of the total structural dead weight. Interior finishing works and installations by tenants have continued even up the end of 1996.

3. Structural modelling

Computer modelling [1] by the architect showed that the core wall resists 90 - 95% of the lateral shear force at lower floors, with the steel framing system sustaining vertical loads and also providing most of the lateral resistance at highest levels. The fundamental mode natural period was estimated as 6.84 seconds (i.e. 0.146 Hz).

The structure was also modelled independently [2] using the SAP90 structural analysis program in two stages. In the first stage, the core wall alone was modelled using shell elements, taking the Young's modulus E as 34 kN/mm^2 for the grade 50 concrete used. The A and B direction natural frequencies were estimated as 0.172 Hz and 0.179 Hz, respectively.

In the second stage, the full framing system was added to the core wall model. Because the resulting model was too large for the SAP90 installation used, the model was split into A-direction and B-direction models in which the translational degrees of freedom in B and A directions, respectively, together with the rotations about A, B and vertical axes were restrained. From these two models, the first three modes in directions A and B (i.e. A1 to A3 and B1 to B3) were identified and are summarised in Table 1.

Table 1: Natural frequencies (Hz) from finite element models

Direction	Mode 1	Mode 2	Mode 3
A	0.185	0.923	2.233
B	0.193	0.961	2.321

The models incorporate the masses of office slab and core slab which are treated as rigid floors. Mass (but not stiffness) of the curtain wall was included but other non-structural

elements e.g. infill panels and water tanks have not been included. Figure 4 shows one of the full-frame models and two B-direction mode shapes.

4. Static performance of the structural system

Measurements of static strain and stress in structural elements at levels 18 and 19 have been made at regular intervals from early 1994 to mid 1996. These measurements [3,4] show interesting patterns of stress and strain in the core wall, CFT columns and floor beams during and after construction. Stress and strain levels in the concrete core wall and one of the large CFT columns increased steadily as expected; Figure 5 shows stress and strain levels at one location in level 18 core wall showing a levelling off of stress after completion of the core wall but a continued increase in strain magnitude. Similar trends are evident in the instrumented columns. For the steel floor beams, the strain readings show small bending moments close to the core wall and more significant hogging moments close to the rigid connections with the CFT columns.

5. Dynamic performance of the structural system during construction

Dynamic response measuring equipment (accelerometers) suitable for measurements of the low level response was available from May 1994. From this time, at intervals of approximately three weeks, recordings of dynamic response in two orthogonal (A and B) directions were made at the highest floor available in the building having a completed office slab.

The accelerometers were typically set up between location '3' in Figure 2 and the furthest edge of the office slab. Natural frequencies and damping ratios were estimated from the ambient response data. From the auto power spectra of the two signals, peaks were identified as possible vibration modes and from the cross power (phase and coherence) spectra these modes were identified as possible torsional or translational modes. These modes were finally identified and characterised during an ambient vibration survey (AVS).

Figure 6a shows the correlation of fundamental translational mode periods with construction progress parameters such as storey level and total mass, and Figure 6b shows the percentage changes in natural frequencies for the lowest two or three lateral modes in each principal direction, and for three torsional modes as construction progressed, using as reference the values measured during the AVS when the building was structurally complete with full water tanks. The translational mode frequencies show a steady decrease towards the reference values while the torsional values pass through minima and then increase slightly to the reference values. The damping values are well scattered and show no obvious trend. However the values average at no more than 1% for all modes.

Particular attention was paid to the period following completion of the main structural elements when the higher sections of the curtain wall were being installed and water tanks were filled. No clear increases in either the stiffness (from natural frequency) or the damping ratio were discernible due to the curtain wall (for the very low vibration levels recorded) while there was a small change in frequencies at the time of filling the water tanks.

6. Prototype test: ambient vibration survey (AVS)

An AVS was conducted over six working days within the period from 20th November to 1st December 1995. The objective of the AVS was to map out the vertical plane mode shapes of at least the first three or four 'translational' modes in each of the A and B directions and also the 'torsional' modes by measuring the horizontal accelerations.

Vertical plane mode shapes were obtained using four accelerometers; one at location '1' (as shown in Figure 2) on level 65 as reference plus up to three others at lower or higher floors, also at location 1. The accelerometers were aligned alternately all in the A direction then all in the B direction recording (digitally) the simultaneous response at the different levels before moving all but the reference accelerometer to different levels. Additionally, on four different levels all four accelerometers were disposed at locations 2 and 3 or at locations 4, 5, 6 and 7 (Figure 2) to determine plan mode shapes i.e. the combination of rigid body rotation and translation for the floor.

The measurements during construction had shown that the fundamental translational modes were not exactly aligned with the A and B axes. The alignment of these and higher translational modes could be determined from the AVS data together with locations of centres of rotation for the torsional modes. Exact definition of frequencies and damping ratios is one objective for a long-term monitoring exercise from which the necessary very long data records should be available.

The signals from the four accelerometers were digitised directly onto a computer hard disk. Data were sampled at 15 Hz (per channel) into 4,096-sample (per channel) data files, with low pass filtering at 2.65 Hz (-3 dB point). Modal parameters were recovered by spectral analysis of the digitised time histories.

7. Results and discussions

For each combination of reference and traveller in each direction, frequency and damping values were obtained for four modes in A and B directions plus four torsional modes. The methodology for processing the response signals and obtaining the mode shapes and frequencies is well documented elsewhere [5] and works on the necessary assumption that the unknown excitation is random and has a slowly varying power spectrum, and that the contributions of other modes to response at the resonant frequency of a specific mode are small. Ideally the modal parameters would be derived while applying a known stationary random input force to the building but this is (except in exceptional cases) impossible. Estimated modal frequencies vary by a only small fraction of a percent between different measurements and mode shape recovery is not affected by varying input so the modal data are valid. The estimated average frequencies (in December 1995) are shown in Table 2.

Table 2: Average frequencies (Hz) estimated from AVS measurements

Direction	Mode 1	Mode 2	Mode 3	Mode 4
A	0.191	0.703	1.55	2.483
B	0.199	0.746	1.73	3.011
Torsion	0.566	1.34	2.31	3.330

Figure 7 shows typical acceleration auto power response from simultaneous measurement in A and B directions at level 65, location '2' (Figure 2). This shows the appearance of translational modes in both A and B directions and the difference in torsional signal strength in the two directions. Since the building is not entirely symmetric it is likely that torsional motion will not be about the geometric centreline and that translational modes will have an element of rotation.

Mode shapes are given in Figures 8 to 10 for the first three A-direction, B-direction and torsional modes based on measurements at location '1' and normalised to unit value at level 65. The solid lines are best fit (degree 6) polynomials. The mode shape values are obtained from averaged cross-power spectra between signals at two positions at the resonant frequency common to both signals. In most cases the spectra are averaged over 16 records and points closely follow the best fit lines, but where fewer or weaker records were used the data points are scattered. For the torsional modes the points are normalised B-direction translational response magnitudes and show greater scatter since the values of φ are sensitive to small errors in accelerometer orientation. Data points for torsional modes at roof level, where there is no core wall, are less than 1.0 (and are not shown) since the accelerometers could not be located at the equivalent location '1'.

8. Response of occupied building

During October 1996 a pair of accelerometers were left at level 65 to record ambient response levels in order to assess the background levels for setting the trigger thresholds of the monitoring system, whose main purpose is to monitor wind response levels. The most significant lateral loads are expected to be due to storm winds and remote earthquakes occurring in Sumatra, Indonesia.

Analysis of signals recorded during two weeks in October 1996 showed that since December 1995, during which time all but a few floors of the building had been occupied by tenants, the natural frequencies had changed by the approximate percentages in Table 3.

Table 3: Percentage of frequency changes due to occupancy

Direction	Mode 1	Mode 2	Mode 3	Mode 4
A	-2.8	-2.6*	-2.6	-2.9
B	-1.7	-1.4	-1.7	-2.9
Torsion	-3.8	-4.5	-2.9	**

* A2 frequency not distinct;
** T4 cannot be recovered.

The strongest response recorded during this period occurred on October 10th at exactly the same time when a magnitude 6 earthquake struck a town in northern Sumatra with an epicentral distance of approximately 700 km from Singapore. While the response to this earthquake was rather small, the record is significant since it is the first building response time history recorded in Singapore.

9. Conclusions

A set of mode shapes up to 3.5 Hz has been identified, comprising modes which are mostly translational and many of which have weak to strong torsion. For all modes, a value of damping ratio approximately 1% was estimated.

The pattern of modes is apparently quite complex due to the variable symmetry of the structure. Finite element models constructed could not predict the torsional response, although the predicted fundamental lateral modes agreed quite well (0.185 Hz predicted against 0.191 Hz measured). Clearly a high level of detail will be required to replicate the observed response.

From the measurements themselves, it can be concluded that the structure is somewhat stiffer than anticipated in the design stage and that for normal conditions the curtain wall has no noticeable effect on the structural behaviour. Details of the core wall layout and of the steelwork connections would appear to be more important factors.

Acknowledgements

References

[1] **Teh, H. S. and Lai, H. P. (1995).** "Structural Aspects of the Republic Plaza". Seminar on Tall Buildings - Design and Construction, 7 July, 1995. The Institution of Engineers, Singapore.

[2] **Chua, K. H., Aw, B. L. (1996).** "Dynamic structural analysis of a tall building". Final Year Project Report ST-06, School of Civil and Structural Engineering, Nanyang Technological University, Singapore.

[3] **Brownjohn, J. M. W., Pan, T. C. and Cheong, H. K. (1996).** "Structural response monitoring of Republic Plaza". Research Report CSE/ST&CO/96-02, Centre for Advanced Construction Studies, School of Civil & Structural Engineering, Nanyang Technological University, Singapore.

[4] **Brownjohn, J. M. W., Inada, Y., Pan, T. C. and Cheong, H. K.** (1995). "Performance of the Republic Plaza structural system during construction". Proceedings, 4th Pacific Structural Steel Conference, Vol. 1, Singapore, 621-628.

[5] **Udwadia, F. E. and Trifunac, M. D. (1974).** "Ambient vibration tests on full-scale structures". Proceedings, 5th World Conference on Earthquake Engineering, Vol. 2, Rome, 1430-1439.

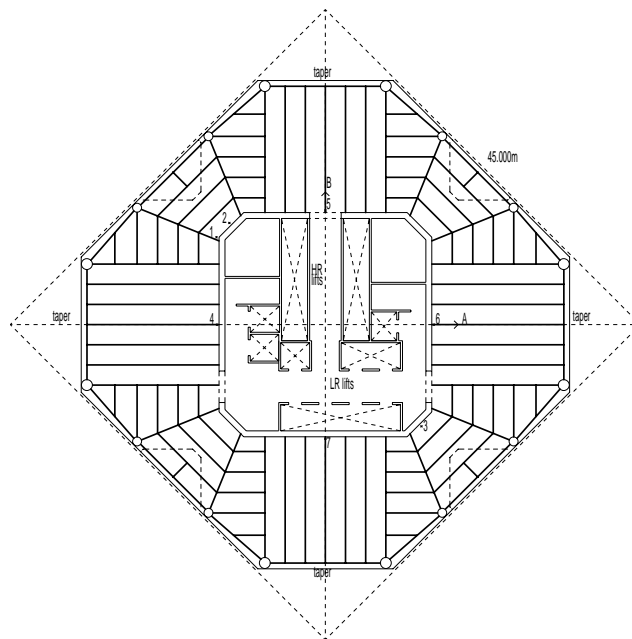**Figure 1: Republic Plaza, showing tapering and non-tapering aspects**

Figure 2: Floor plan at level 19 (with A and B directions and accelerometer locations)

Figure 3: View of levels 43 to 45 showing rigid connections at columns, simple connections at core wall (to the right) and steel decking for composite office slab

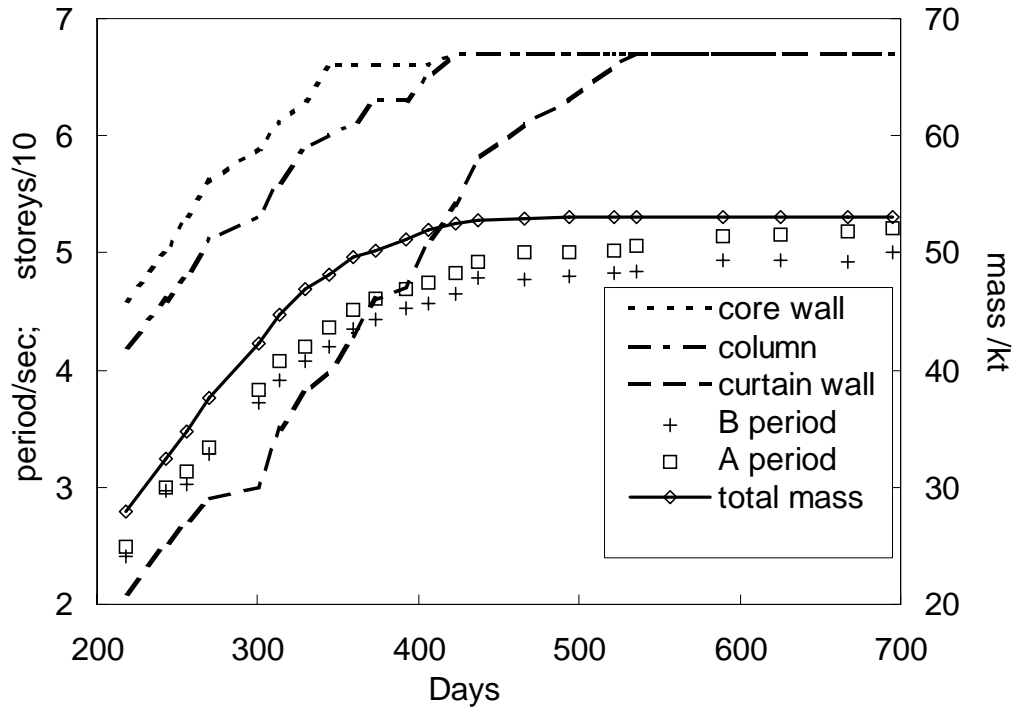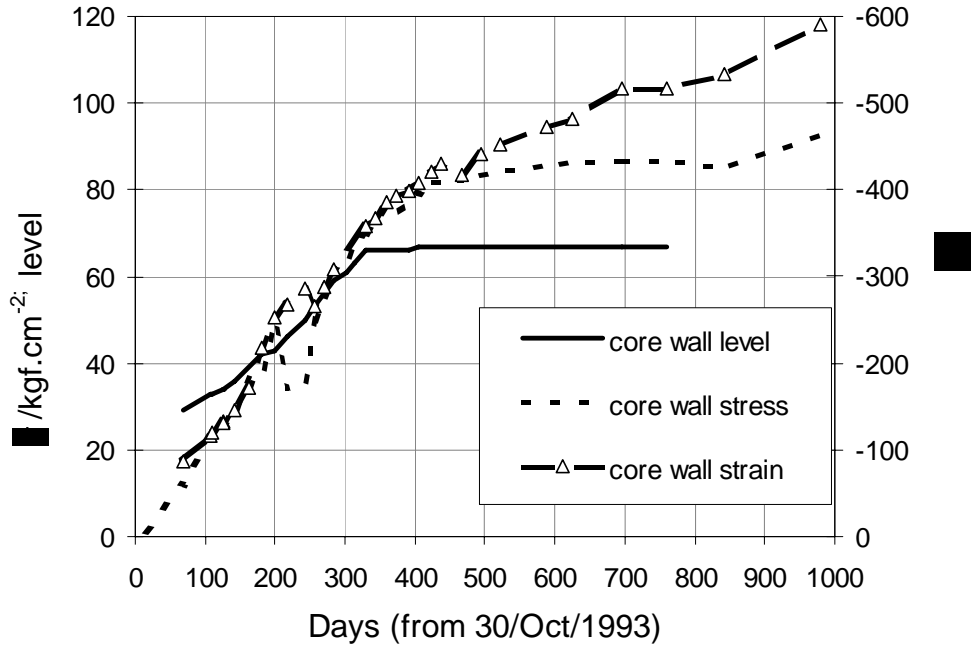**Figure 4: Finite element models and core wall components of modes B1 and B2**

Figure 5: Stress and strain readings in level 18 core wall and construction progress

Figure 6(a): Variation of modes A1 and B1 vibration periods measured during construction
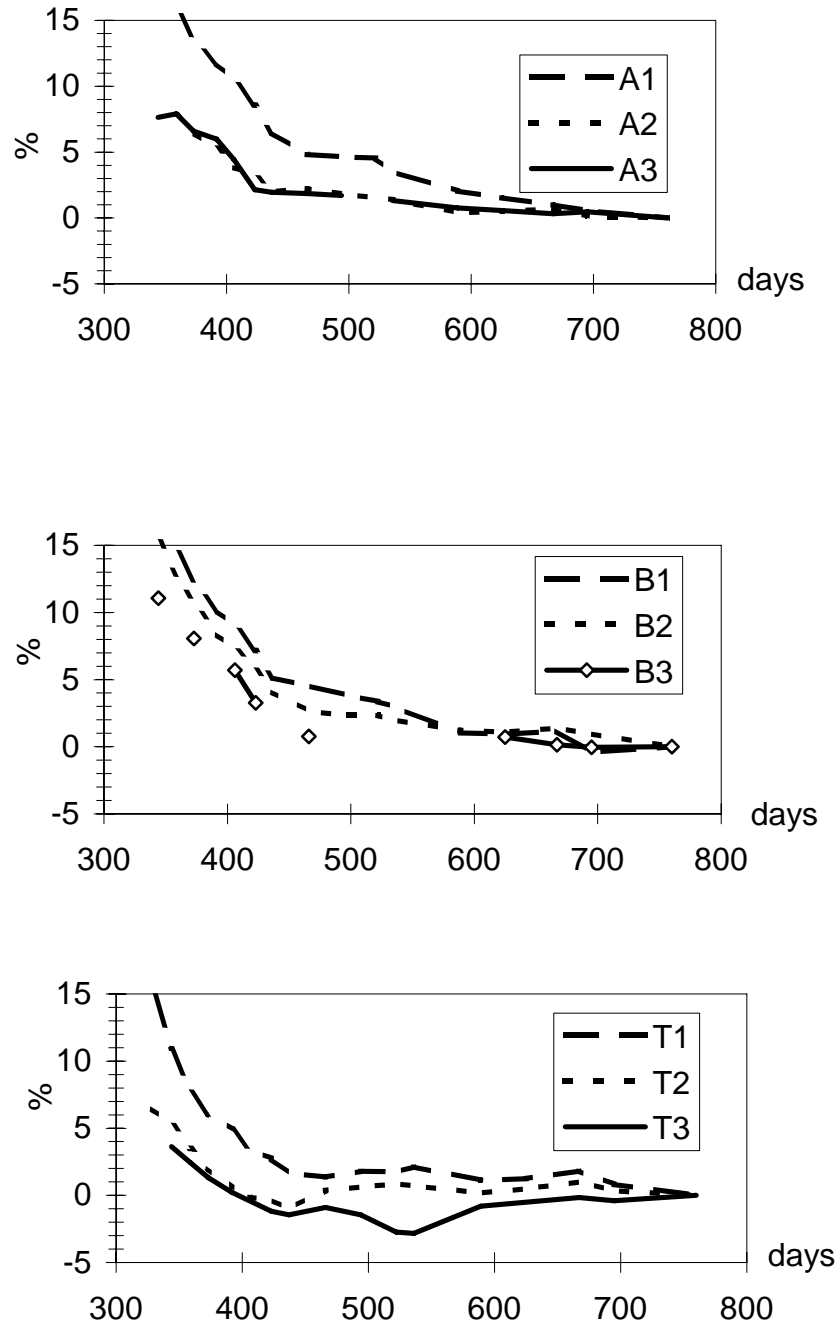
Figure 6(b): Variation of modes A1-A3, B1-B3 and T1-T3 vibration frequencies measured as percentages of values obtained during AVS

11
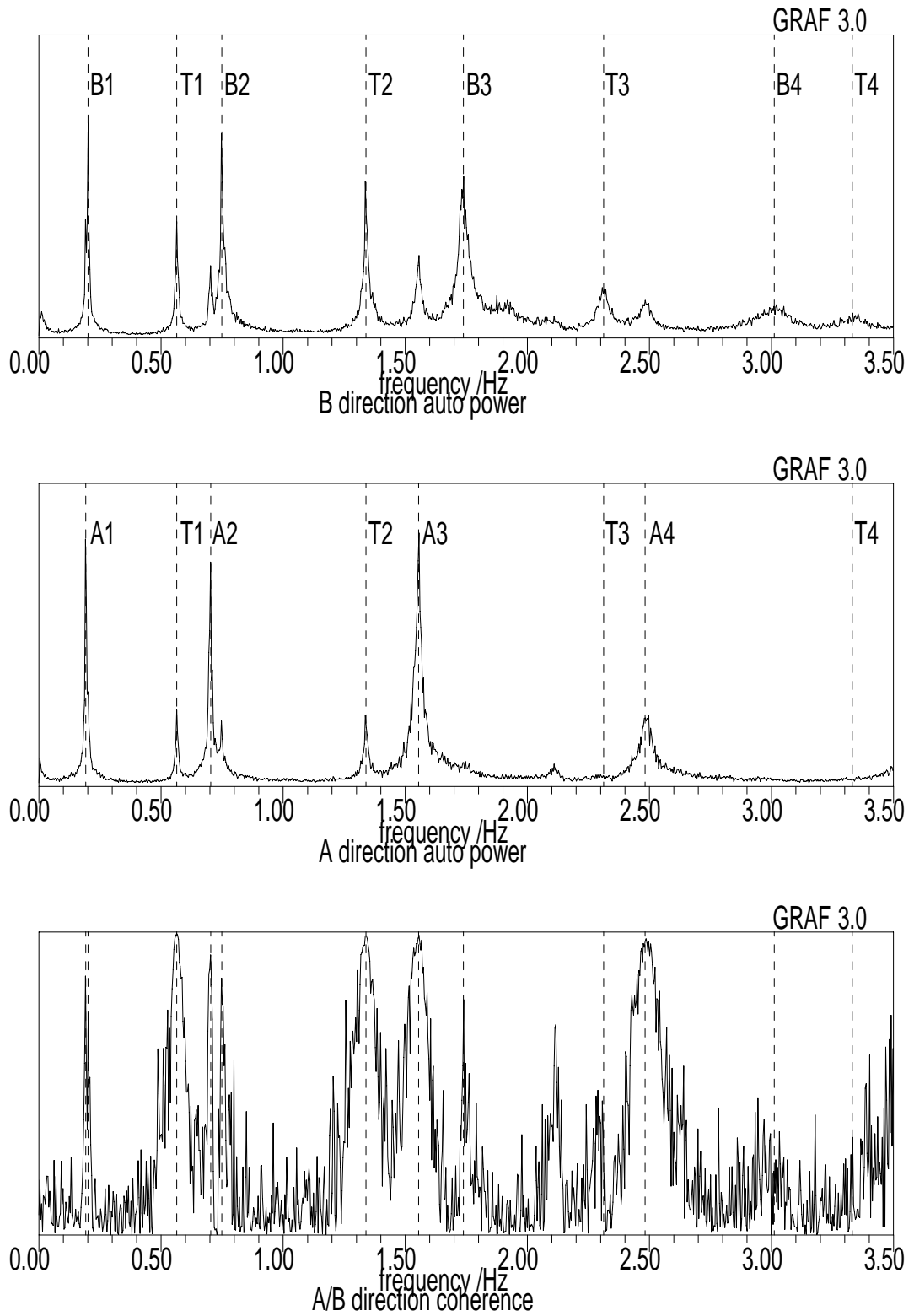
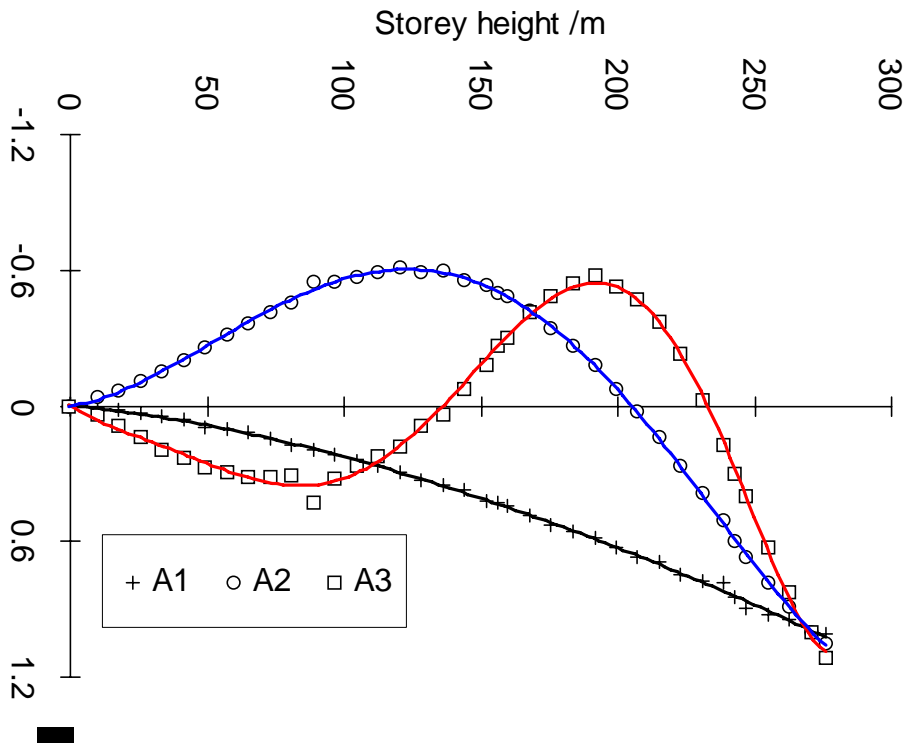**Figure 7: Frequency spectra from A and B direction measurements at location 1 on level 65**
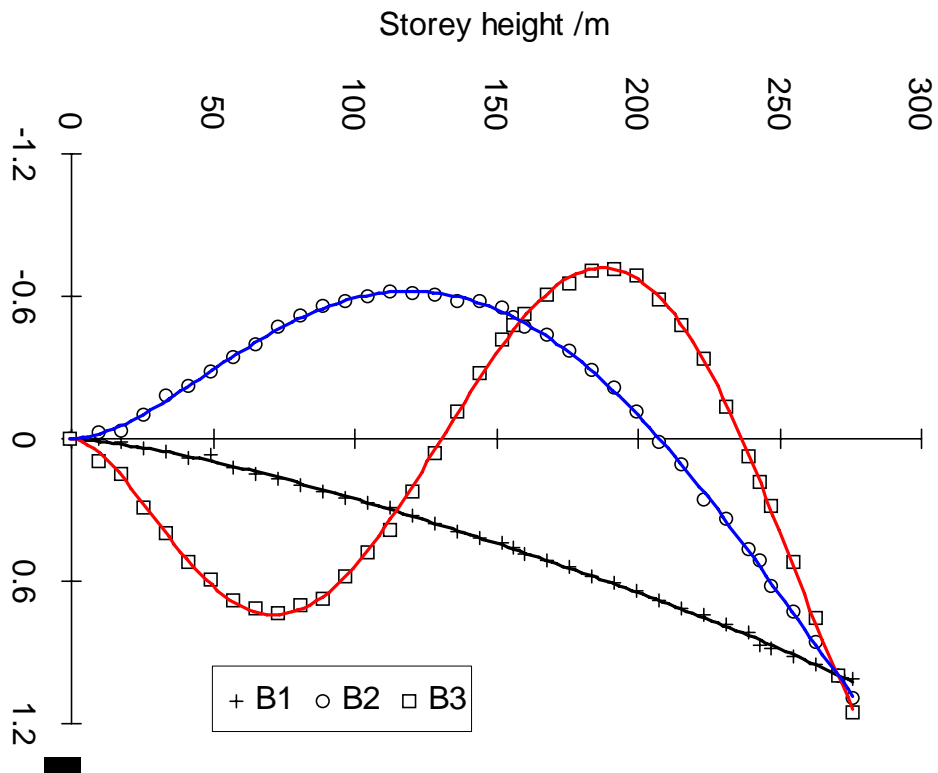
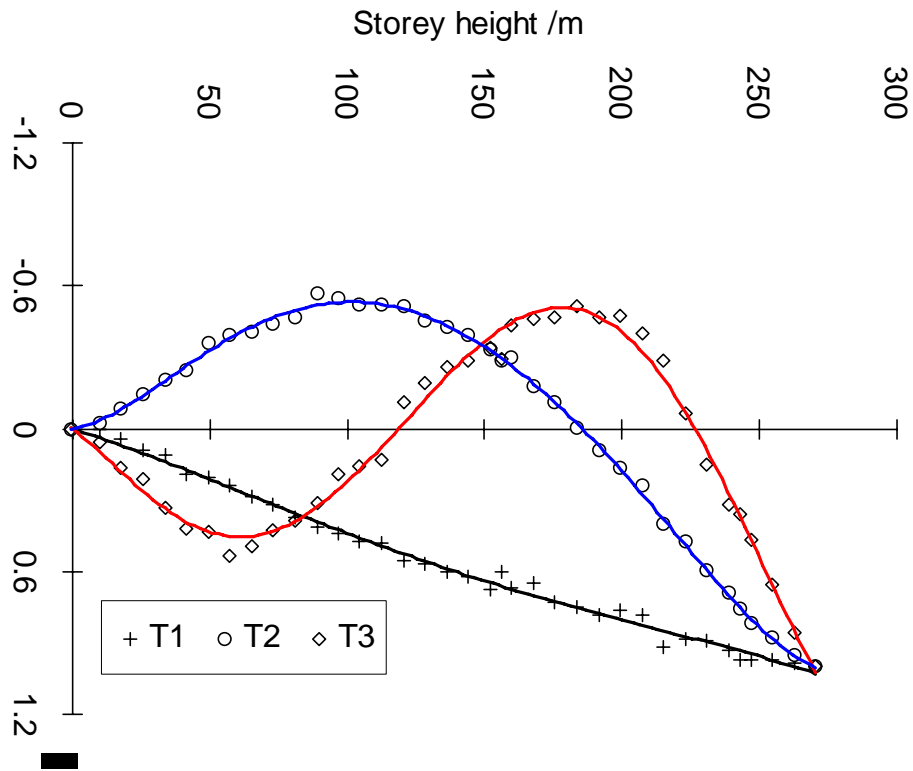**Figure 8: Mode shapes in A-direction**

Figure 9: Mode shapes in B-direction

Figure 10: Torsional mode shapes (B-direction component)