

LONG SPAN STEEL PEDESTRIAN BRIDGE AT SINGAPORE CHANGI AIRPORT

PART 1: PREDICTION OF VIBRATION SERVICEABILITY PROBLEMS

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ABSTRACT

Changi Mezzanine Bridge is a 140m span flat arch footbridge constructed from welded tubular steel sections inside a tunnel that connects two passenger terminals at Changi Airport, Singapore.

A series of vibration measurements were made on the bridge during construction, showing that non-structural cladding added mass and reduced the natural frequencies while also increasing the modal damping, from as little as 0.2% originally to around 0.4% for critical vibration modes.

From these preliminary studies leading up to the opening of the bridge in early 2002, it was clear that the first symmetric lateral vibration mode (LS1) at approximately 0.9Hz and the first symmetric torsional vibration mode (TS1) at approximately 1.64Hz could be excited easily by pedestrian movement. The modal parameters for mode LS1 suggested that the bridge could suffer from synchronous lateral excitation for a walking pace of 1.8Hz while for TS1 the potential problem was the coincidence of the mode frequency with the lower range of predominant footfall frequencies together with a very low modal mass.

These possibilities had been identified by the consultant who advised that an experimental study of the characteristics of low frequency vibration modes should be conducted to check vibration serviceability predictions based on analytical modeling. Forced vibration testing using a combination of shakers and humans was used to determine in a very short time scale, the properties for modes below 3Hz. The mode shapes and frequencies compared favourably with predictions from the consultant's finite element model that had been used to show that with a large number of pedestrians, comfort levels would be exceeded and the bridge would be unserviceable.

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1 INTRODUCTION

The lively nature of footbridges, particularly those that are cable supported, has made them an enduring subject for research by structural engineers. In one of the earlier scientific studies (1) students were used to excite vibrations of suspension bridges in order to estimate the damping capacity, a procedure that has since been applied more systematically (2) and (with some refinement) in the work reported here. While a degree of liveliness may be tolerated and even enjoyed on a bridge in a theme park or tourist attraction (3), acceptable vibration limits need to be specified for a bridge to remain serviceable under normal use. For example, the UK design code for bridges, in the form of BD37/01 (4) specifies, for footbridges having fundamental frequencies below 5Hz, the limiting accelerations in terms of natural frequency for a single pedestrian and goes on to show how to estimate both natural frequency and response levels. Most current design codes dealing with vibration serviceability of footbridges (5) suggest either to avoid ranges of natural frequencies matching natural walking paces or to check simulated response to walking against frequency-dependent acceleration levels.

There are some clear deficiencies in these codes for predicting serviceability. Of necessity they are simplistic, there are questions on the reliability of the human walking models used, serviceability with more than a single pedestrian is not specifically addressed and lateral vibrations have only a brief mention in the most recent codes (4). A large body of research on walking loads is now available (6), but for crowd loading, research has usually been confined to floors of relatively high natural frequency with people walking or jumping (7,8). Also, until the recent experience with the London Millennium Bridge (9), little was known about the level and effect of lateral walking loads on footbridges.

Hence for a long span steel footbridge designed before the Millennium Bridge experience but completed in early 2002 (10), having fundamental natural frequencies in lateral and vertical vibration below 2Hz and expected to operate with a normal maximum loading of around 100-200 pedestrians, there is likely to be uncertainty and a degree of concern about vibration serviceability.

Analytical vibration serviceability studies (11) were undertaken by the consultant in the light of the Millennium Bridge experience, and this paper describes experimental work undertaken at their suggestion in order to identify the dynamic characteristic of the bridge and validate the predictions of response to pedestrian loads.

By chance, a student project had provided a series of preliminary observations on changing dynamic properties during construction, and as these supported the consultant's predictions it was logical to extend the studies into a formal modal test to extract modal parameters so as to validate the consultant's finite element modeling and serviceability predictions. These predictions, which indicated that the vibrations for a reasonable number of pedestrians would exceed comfort limits and render the bridge unserviceable, are presented briefly in the final part of the paper.

The final outcome of the work presented in this paper is that the analytical model had been proven correct, yet the nature of the forcing functions due to crowd loading was still uncertain. Hence it was deemed necessary to conduct a controlled walking test with a large number of pedestrians, essentially to check the loading mechanisms assumed in the predictions. The walking 'proof test' is reported separately (12).

2 CHANGI MEZZANINE BRIDGE

Changi Mezzanine Bridge (CMB) was opened in early 2002 and provides for an underground walkway between two terminals at Singapore's Changi Airport situated north-west and south-east of an expressway. A tunnel underneath the expressway contains the bridge and also houses the airport mass rapid transit (MRT) terminus. MRT tracks approach from the north-west and the platforms run parallel to the bridge at its foundation level.

The bare frame (skeleton) of the bridge comprises approximately 8×10^5 kg of cold-formed structural steel circular hollow sections (CHS) and rectangular hollow sections (RHS) welded together to form a shallow arch. The arch is supported on a pair of pins at each end of the 140m main span; back spans of 30m are simply supported and the entire structure is clad in approximately 3000m^2 of semi-opaque glass panels up to 22.5mm thick.

Along each side, top and bottom CHS chord members are connected by a sequence of concentric braces, likewise the bottom pair of CHS chord members are linked by concentric braces to form the bottom arch belt. Transverse K-bracing systems at 7.6m centres in the spanwise direction connect each side and provide torsional stiffness. The deck level has a more complex arrangement of CHS and RHS that provide support for the moving walkways (travelators) and glass floor panels. There are large openings around the pins to allow for the escalators rising from deck level and a diamond-shaped hole through the deck at centre span for aesthetic purposes. The deck level walkway is partially cantilevered from the top chords using a triangular framing system.

Figure 1 shows a half elevation and half section plans on the floor deck and arch belt, identifying the locations of escalators, travelators and openings. Figure 2 shows a view of the partially complete steel skeleton frame while still supported on props, including (in the distance) one of the transverse K-braces. Once the frame was structurally complete, the props were removed (in June 2001) and an elaborate steel framing system to support the cladding, walkways and travelators was added. Glass cladding was added progressively, first to the sides, then to the walkway level and soffit. Figure 3 shows the completed bridge from the MRT platform level.

At mid-span the frame is 7.6m wide and 1.2m deep, at the pin it is 11.6m wide by 5.8m deep; the height of the arch is 6.75m. According to the consultant (11), the bridge mass is 6.5×10^3 kg/m i.e. 1.3×10^6 kg total including all fixtures and fittings and includes approximately 2×10^5 kg of glass, although the final value of as-built mass could not be obtained. The total deck area available for pedestrians, excluding voids and travelators is approximately 840m^2 .

3 EARLY VIBRATION STUDIES OF THE BRIDGE

Vibration modes of a structure may be characterized by modal frequency, shape, mass and damping and are reflections of mass and stiffness distribution and capacity to dissipate energy. Mode frequencies in particular are very sensitive to boundary conditions, and the original motivation for studying CMB was to investigate the translational stiffness of massive pin bearings by inference from the mode frequencies. During 2001 a series of visits with students provided an opportunity to investigate the bearing stiffness, but because it was also possible to observe the construction progress, the objective of the study changed to tracking the changing dynamic characteristics and ultimately to validating the consultant's serviceability studies.

In a series of measurements beginning after the de-propping of the completed skeleton frame, the frequencies and damping ratios of the lowest vertical and torsional vibration modes were tracked. The initial measurement was made on the skeleton frame using impulsive response to single jumps or hammer hits on the top chord. Due to safety constraints and difficult access, only a few modes could be excited and damping could not be estimated. As accessibility improved, it was possible to reach the deck level and measure response at several locations so as to provide some indication of mode shapes, and also to record lateral response. Once a usable walkway had been installed, it became obvious that strong and clear vibrations could be generated by normal walking, and so the last of these 'ad-hoc' measurements used jumping, walking and swaying timed using a metronome to generate strong signals whose free decay could provide reliable estimates of modal damping as well as frequency. Other methods and tools such as an instrumented hammer and pulling on a rope slung around a chord were tried, but the human movements were the most effective. Surprisingly, measurements of vibration response at the pin failed to show clear signals, indicating very rigid foundations.

Tables 1 and 2 show the frequencies and damping ratios estimated from the measurements during construction. As construction progressed and it was possible to get clearer results, more modes were visible in the vibration signals, e.g. as distinct peaks in the Fourier (auto power) spectra. The nature of the modes was also identified progressively through relative amplitudes of signals from accelerometers, and up to the formal vibration testing described later, seven modes were identified. In fact there are eight modes up to 3Hz and the sixth in the sequence was identified in the subsequent testing.

From the beginning it was clear that the simplest torsional vibration mode could be readily excited. This turned out to have a deformed shape or mode shape symmetric about a transverse axis of symmetry through centre of the

bridge and with rotational about the centerline along the span. This mode was designated TS1: T for torsional, S for symmetric, 1 for the first in a sequence of symmetric torsional modes. Likewise, a lateral mode with motion symmetric about the transverse axis of symmetry and designated LS1 was identified during normal walking. In Table 1 antisymmetric modes with nodes of zero motion at the transverse symmetry axis are designated A e.g. LA1. The second symmetric lateral mode LS2, was not picked up in the early measurements and has nodal points of zero motion symmetric about midspan.

Some interesting observations arise from these data. The first, and unsurprising, result is the reduction in natural frequency, as non-structural mass was added. Assuming an unchanged stiffness and a uniform distribution of both steel and glass masses leads to estimates of mass increase between 40% and 58%. Given that the steel fabricator supplied approximately 8×10^5 kg of steel for the skeleton frame, and taking the symmetric vertical VS1 as most reliable and representative of mass distribution suggests a final total mass around 1.2×10^6 kg.

Secondly, the measured frequencies, representing the lowest modes all (except for VS1) were found to be in the range that could be excited by normal walking pace rates i.e. from 1.5Hz to 2.5Hz (13).

Thirdly, not only is the increase of damping values remarkable, but also, the values observed are rather low, even for the completed clad structure. The low values of damping for TS1 are reliable, having been obtained by observing the slow free decay of the mode from strong response.

Figure 4 shows mode shapes estimated in early January 2002 using a 'response only' system identification procedure (14) applied to two lateral and four vertical acceleration signals from locations arranged symmetrically about the two horizontal axes of symmetry through the bridge mid-point. The mode frequencies indicated are slightly less reliable than those obtained from the free decay of a single mode, but the indicated mode shapes are consistent with those obtained from later measurements.

4 MODAL MASS ESTIMATION USING HUMAN EXCITERS

Frequencies, mode shapes and damping ratios do not suffice for prediction of vibration response to applied loads. The structural mass must be known, more specifically the effective mass of the bridge for a particular vibration

mode. For the normal mode analysis procedure where the behaviour of a dynamic system is taken as the sum or response of decoupled or ‘normal’ modes, the term modal mass is used to describe the scale factor that applies for mode shape amplitudes together with response calculations for mode r . Modal mass is defined by the product of mass with squared mode shape integrated over the entire structure, i.e. for mode r :

$$m_r = \oint \psi^2 dm \quad \text{Equation 1}$$

The definition used here is that the mode shapes are scaled to have unit value (1m) for maximum amplitude (at point k) i.e. $\left| \psi \right|_{\max} \equiv 1$ so that for harmonic excitation and response at point k in mode r , the bridge behaves as a single degree of freedom oscillator with mass m_r . Hence it should be possible to estimate m_r from co-located force and response measurements and to then use the value in response predictions.

In the preliminary testing, modal mass estimates were obtained via off-line measurements with a non-portable laboratory force plate. Force signals for volunteers jumping or swaying at frequencies corresponding to specific modes were recorded on the force plate in the laboratory then the same jumping and swaying was repeated at the bridge to excite the modes. Figure 5 (top) shows the response of the bridge due to a single person swaying at 1.8Hz close to the midspan of the bridge to excite mode LS1; for the first few cycles a steady increase in acceleration of 0.336mm/sec^2 per cycle is observed; when it seemed the maximum response had been reached the vibrations were allowed to decay freely. Figure 5 (middle) shows the force time series of the same swaying, recorded using the force plate.

Figure 5 (bottom) shows the simulation of the effect of the recorded force signal on a 1000kg oscillator having damping and frequency values obtained from the free decay of the bridge response; the increase per cycle is 144mm/sec^2 . Since the ratio of acceleration increase per cycle between bridge and 1000kg simulation is 428, the bridge modal mass should be approximately of 4.28×10^5 kg.

The estimate is subject to errors in the variability of the swaying forces, the extra mass added to the bridge after this exercise and the reduction in mode shape due to not exciting and measuring at exact midspan. The latter two factors

are in opposite senses so the figure gives a rough estimate of the final modal mass for LS1. By a similar procedure used with jumping it was possible to estimate mode TS1 modal mass as $1.45 \times 10^5 \text{kg}$.

5 MODAL SURVEY FOR IDENTIFYING MODES FROM 0Hz TO 3Hz

Both the preliminary measurements and the independent calculations by the consultant (11) using prior finite element analyses indicated that modes LS1 and TS1 could be problematic for vibration serviceability under pedestrian loading. An early measurement of response to several pedestrians is given in Figure 6 which shows a time-frequency plot (spectrogram) of lateral (above) and vertical (below) response close to midspan while 20-30 construction workers crossed the bridge generating vertical and lateral response of up to 0.05m/sec^2 . These signals, which were used to obtain the mode shapes of Figure 4, represent a broadband excitation yet LS1 and TS1 dominate the response in respective directions.

Further experimental investigations were suggested to check the validity of the consultant's predictions (11), requiring experimental estimates of damping, frequency and modal mass of the completed structure for the first eight vibration modes.

Hence, at the suggestion of the consultant, Singapore Land Transport Authority (LTA) commissioned vibration measurements in two phases to validate the finite element models used for the serviceability predictions and then to study full-scale response in a controlled walking test with a large number of pedestrians. Phase 1 was a full-scale vibration survey to identify vibration modes up to 3Hz. Phase 2 was a 'proof test': a set of pedestrian walking tests reported separately (12).

6 OUTLINE PLAN FOR PHASE 1 VIBRATION SURVEY

The most acceptable estimates of modal parameters can be obtained where controllable and measurable excitation force can be applied. As the previous discussions illustrate, knowledge of the force is necessary for estimating modal mass. Testing of the London Millennium Bridge (15) had determined that forced vibration using a combination of hydraulic and rotating mass shakers was necessary to provide forces of the order of 1kN to resolve the vibration modes above any level of background noise.

No such exciters were available for CMB testing, but small long-stroke electro-dynamic shakers were available having maximum available output of up to 200N amplitude for the range 1-3Hz. ‘Virtual testing’ using the estimated modal masses and damping ratios of the bridge showed that given a low level of background noise, sufficient response should be generated for a clear resolution of all vibration modes.

A set of twelve force balance servo accelerometers was also available and a grid of measurement points was identified for mapping mode shapes in both lateral and vertical directions using two consecutive arrangements of accelerometers at locations as indicated in Figure 1 equally spaced at intervals of 15.2m with BP at midspan:

Test sequence 1 BK, BB, BD, BF

Test sequence 2 BK, BH, BM, BP

At each location a trio of accelerometers, one on each side of the deck measuring vertically and one on the east side measuring horizontally was to be used, retaining one trio at BK as a reference for normalization of mode shapes.

For each configuration the first stage of testing would use a chirp signal to obtain a set of frequency response functions (FRFs). In the second stage, resonance would be established by sine excitation at each mode frequency in turn so as to first observe the mode shapes at steady state, and then to observe free decay with the force set to zero.

The FRFs would provide estimates of all four modal parameters, as follows.

7 FREQUENCY RESPONSE FUNCTIONS

The ratio of acceleration response at location j to force input at location k in steady state vibration at frequency ω is defined by the acceleration or inertance form of the FRF (16) having units of mass⁻¹

$$H_{jk}(\omega) = \ddot{X}_j(\omega)/P_k(\omega) = \sum_{r=1}^n \frac{{}^r\psi_j {}^r\psi_k}{m_r} \cdot \frac{-\omega^2}{-\omega^2 + \omega_r^2 + 2i\zeta_r\omega_r}. \quad \text{Equation 2}$$

where $\ddot{X}_j(\omega)$ is acceleration at location j

$P_k(\omega)$ is force at location k

c_r is mode r damping constant

k_r is mode r stiffness

$\zeta_r = c_r/2m_r\omega_r$ is mode r damping

$\omega_r = k_r/m_r$ is mode natural frequency

${}^r\psi_j$ is mode shape for mode r at location j so that $\max |{}^r\psi_j| \equiv 1$,

${}^r\psi_j {}^r\psi_k / m_r = {}^rA_{jk}$ is the modal constant for the force/response pair j,k and

n is number of modes participating in the response

The circle fit method (16) was chosen to extract modal parameters. Where $j=k$ and ${}^r\psi_k = 1$ the ‘driving point’ modal constant is the inverse of the modal mass m_r and the values of modal constant for fixed driving point and varying j provide the mode shape.

8 MODIFIED PLAN FOR VIBRATION SURVEY

8.1 Stage 1: Measurement of FRFs using chirp signals

The full-scale testing activities for the vibration survey were limited to approximately six hours. The unavoidable presence of a small number of technicians working on mechanical equipment on the bridge generated ‘noise’ which contributed to a difficulty in obtaining reliable FRFs at frequencies below 1.5Hz, and the best FRFs were obtained during meal breaks. Figure 7 shows one of the better ‘driving point FRFs’ for lateral excitation with a 204.8 second narrowband chirp signal. From such data it was possible to estimate natural frequencies and driving point modal masses via circle fitting only for modes with higher frequencies or lower mass i.e. TS1, LS2 and TA1. Better quality FRFs would have required either a more slowly varying chirp signal bypassing frequency ranges of no interest or stepped sine testing that was used to good effect in the longer time scales of LMB full scale vibration study (15).

8.2 Stage 2: Steady state sine excitation

Steady state excitation at estimated modal frequencies was used for extracting mode shapes. Mode shapes were obtained by comparing amplitudes and phase angles of response signals narrow-band filtered around the excitation frequency. Further estimates of modal mass were available from studying the force and response signals, for example as shown in Figure 8 for mode LA1. Using the same procedure that had been applied to the swaying response data of Figure 5, modal masses obtained from the initial build of the shaker-induced response were obtained for a number of modes, as shown in Table 3. Frequencies and damping ratios obtained from the free decay correspond to very low levels of response and are also given in Table 3 under the headings ‘sine’.

8.3 Stage 3: Human excitation

With the good experience of using human forcing in the preliminary tests, the same technique was repeated to excite the set of eight vibration modes by ‘swaying’ and jumping at each frequency in turn using a metronome then studying the free decay. Decaying exponentials constrained to have the same frequency and damping but different phases and amplitudes were fitted to each response channel to obtain mode shapes as well as a common damping and frequency. While the shaker generally provided a clearer response for higher frequency modes, the jumping/swaying was more effective for the lower modes.

9 SUMMARY AND COMPARISON OF EXPERIMENTAL MODES WITH FEM MODES

Figure 9 shows the six modes identified clearly from free decay of response to human excitation, and Figure 10 shows the two modes obtained only from the steady state shaker testing. Time did not permit measurements on the other half-span, so displacements were reconstructed using the symmetry or asymmetry evident from the preliminary measurements as shown in Figure 4.

Table 3 summarises the modal parameters obtained from the different methods. The data in bold are the final values from testing that are believed to be most reliable and correspond to small amplitude vibrations. Modes LS1 and TS1 are the most reliably estimated and important modes.

Two sets of FEM predictions were available, one from the consultant's model for serviceability assessment and setting the bridge total mass at 1.3×10^6 kg, the other developed and refined by LTA to study deflections due to dead loads and setting bridge mass at 10^6 kg plus applied loads to account for the non-structural cladding. LS1 is predicted well by both models, otherwise the consultant's model generally underestimates the frequencies while the LTA model overestimates them, because the weight but not the mass of the cladding is included. Both models switch the order of VA1 and LA1 with respect to the prototype values, but generally the errors are acceptable. The modal mass estimates from the consultant's FEM should be most reliable and the value for TS1 is the lowest, explaining the ease with which the mode is excited.

The two-dimensional nature of modes LS1 and TS1 are shown in Figure 11 which compares the LTA FEM modes with those obtained experimentally; the circles represent the experimental modal ordinates and it is clear that mode LS1 has an element of torsion while TS1 has a significant lateral component.

The conclusion is that the consultant's FEM was sufficiently accurate in terms of frequency, mass and mode shape. and that provided the forcing functions and damping ratios assumed are valid the serviceability predictions should be realistic.

10 NON-LINEARITY AND DAMPING

The frequency and damping values given in Table 3 are for low to medium levels of response amplitudes.

Examination of decay traces with piecewise curve fitting over short intervals shows a clear decrease of frequency and increase of damping for higher amplitudes, as shown in Figure 12 for modes TS1 and VS1. For LS1 there is a very weak dependence of frequency on amplitude but damping values show no clear pattern. The consultant had assumed damping values of 0.7% for all serviceability predictions, an assumption based on experience and best information available but appearing not to have been sufficiently conservative.

11 SERVICEABILITY PREDICTIONS

According to the consultant (11) acceptable limits established for comfort are in the range 0.2-0.4m/sec² due to lateral vibrations and for in the range 0.5-1.04m/sec² for vertical vibrations.

The bridge loading code currently used in Singapore is BD37/01 (4) which specifies allowable vertical accelerations of

$$a = 0.5\sqrt{f_o} \quad \text{Equation 3}$$

equal to 0.48m/sec² for CMB, a level not achieved during any of the tests.

No loading model is available for lateral response, but for vertical response the limiting value from equation (3) is deemed to be due to a single perfect pedestrian moving along the bridge at $v_t = 0.9\sqrt{f_o}$ m/sec while generating a pulsating load of

$$F_1 = 180 \sin 2\pi f_o t \text{ N.} \quad \text{Equation 4}$$

Using the measured modal parameters the calculated peak response level would be 0.102m/sec² which is acceptable.

This was regarded by the consultant to be a minimum requirement given that one of the issues with current loading codes issues identified by the experience with LMB is the probability that vertical forces (if not the resulting response) for such a bridge would be far greater than the ‘single perfect pedestrian’ provision.

The post-Millennium Bridge study (11) by the consultant using the FEM that was subsequently validated included a range of simulations for different pedestrian loading scenarios for all eight vibration modes described in this paper. Although up to 1680 pedestrians could manage to stand on the bridge, the most plausible loading scenarios are P_p where a practical maximum of 650 pedestrians are moving in four lines with 0.8m spacing between pedestrians, and P_{200} where a daily maximum value of 200 pedestrians are on the bridge with random spacing.

The analyses were based on dynamic loads P for N pedestrians obtained via the formula

$$P = (\alpha \times N \times F_1) \times A \quad \text{Equation 5}$$

where α is a correlation coefficient representing the proportion of pedestrians in step and was taken as $\alpha = 0.4$ for all modes, leading to predicted response values in m/sec^2 :

	LS1	TS1
P_p	1.3	11.5
P_{200}	0.4	3.6

The value of α was based on the evidence from LMB studies that 40% of pedestrians were ‘locked in’ to synchronous lateral excitation, although extension to vertical excitation is so far unproven. Accelerations exceeding 1g are physically impossible and as with LMB, pedestrians would stop moving well before such values could be achieved. Even with these reservations there was concern that high levels of response in either mode could be observed in the prototype, and on the basis of the information available at the time of the consultant’s study they recommend a pedestrian loading test’ on the bridge.

Subsequent studies by the consultant for LMB (9) had produced the formula for predicting the critical number of N of pedestrians distributed evenly across the bridge required to induce synchronous lateral excitation (SLE):

$$N = \frac{8\pi f_r \zeta_r m_r}{k}.$$

Equation 6

Given that the constant k was estimated as 300Ns/m, a figure of $N=247$ results from the consultants predictions, whereas based on the values obtained from the prototype testing, the figure would be $N=145$. Clearly there was a need to check this number and take necessary steps to prevent SLE for such a small number of pedestrians.

12 DISCUSSION AND CONCLUSIONS

A set of eight vibration modes up to 3Hz for Changi Mezzanine Bridge has been identified. The damping values, even with cladding, are very low, as small as 0.4%, depending on amplitude, while there is also a dependence of frequency on amplitude. It is observed that fundamental lateral and torsional modes LS1 and TS1 are the most strongly excited by normal walking, and that almost all vibration modes up to 3Hz can be excited by timed jumping or swaying. Several procedures were used to excite and analyse the vibration modes and a range of estimates of parameters are recorded.

The study of CMB has some parallels with the earlier investigation of the dynamic characteristics of London Millennium Bridge, which has the same span and same construction material, but is three times heavier and located in a closed environment. For reliable estimation of FRFs from LMB a powerful shaker was to provide high signals against background wind-induced ambient noise. Given a very tight schedule a different approach to excitation was employed at CMB.

For the bridge itself the testing confirmed that even for high response levels, damping remained rather low. Validation of the analytical FE models provides credibility for predictions of excessive response in the first torsional and lateral modes TS1 and LS1, yet there remains a doubt about the validity of the loading models. Furthermore, separate predictions suggested that as few as 145 pedestrians could induce synchronous lateral excitation (SLE). Hence there was a clear need for a ‘proof test’ with a large number of pedestrians to check the predicted effect of crowd loading and test for SLE. The procedures and results of the proof testing are reported in a separate paper together with the vibration mitigation measures adopted to control lateral vibration response.

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REFERENCES

- 1 Selberg A, 'Dampening effect in suspension bridges' IABSE Publications **10** 1950, 183-198.
- 2 Wheeler JE, 'Prediction and control of pedestrian-induced vibration in footbridges', ASCE Journal of Structural Engineering **108**(9) 1982, 2045-2065.
- 3 Brownjohn JMW 'Vibration characteristics of a suspension footbridge' J Sound and Vibration **202**(1), 1997, 29-46.
- 4 BD37/01 British Standards Institution, Steel, concrete and composite bridges: Specifications for loads, Part 2, Appendix C. BS5400, 1978, British Standards Institution.
- 5 Pimentel RL, Pavic A, Waldron P, 'Evaluation of design requirements for footbridges excited by vertical forces from walking' Canadian Journal of Civil Engineering **28**, 2001, 769-777.
- 6 Pavic A, Reynolds P, 'Vibration serviceability of long-span concrete building floors. Part 1: Review of background information', The Shock and Vibration Digest, **34**(3) 2002, 187-207.
- 7 Ellis BR, Ji T, 'Loads generated by jumping crowds: experimental assessment' BRE IP4/02
- 8 Ellis BR 'On the response of long span floors to walking loads generated by individuals and crowds' The Structural Engineer **78**(10), 2000, 17-25.
- 9 Dallard P, Fitzpatrick AJ, Flint A, Le Bourva S, Low A, Ridsill Smith RM, Willford M, 'The London Millennium Footbridge' The Structural Engineer **79**(22), 2001, 17-33.
- 10 Roche M, 'Performance in lieu of prescription in bridge design – Changi Airport mezzanine bridge, Singapore' Transportation Research Record **1770**, 2001, 195-203.
- 11 Roche M, Land Transport Authority Changi Bridge Vibration Characteristics, Ove Arup & Partners USA, December 2000.
- 12 Brownjohn JMW, Fok P, Roche M, Moyo P, Long span steel footbridge at Changi Airport. Part 2: Crowd loading tests and vibration mitigation measures
- 13 Bachmann H, Ammann W, Vibrations in structures induced by man and machine. Structural Engineering Documents 3e, IABSE, Switzerland, 1987.
- 14 Juang J-N, Pappa RS, 'An eigensystem realisation algorithm for modal parameter identification and model reduction', AIAA Journal of Guidance, 1985, **8**(5), 620-627.
- 15 Pavic A, Armitage T, Reynolds P, Wright J, 'Methodology for modal testing of the Millennium Bridge, London', Proceedings ICE, Structures and Buildings **152**(2), May 2002 111-121.
- 16 Ewins D J, 'Modal Testing. Theory, practice and application', Second Edition, Research Studies Press Ltd, 2000.

Table 1 Frequencies /Hz estimated from vibration free decay during construction

Measurement		i	ii	iii	iv	v	vi
mode	name	June/01	Aug/01	Oct/01	Nov/01	Dec/01	Jan/02
1	LS1	-	-	1.05	0.95	0.890	0.897
2	VS1	1.36	1.34	1.31	1.20	1.128	1.114
3	VA1	1.67	1.59	1.61	1.42	-	1.408
4	LA1	-	-	-	-	1.461	1.471
5	TS1	2.05	2.04	2.01	1.75	1.64	1.632
6	LS2	-	-	-	-	-	2.510*
7	TA1	-	-	-	-	-	2.690
8	VS2	3.22	2.95	3.12	2.82	2.73	-

Mode types

L= lateral V= vertical T= torsional
 S= symmetric A= anti-symmetric

Construction state

- i just unpropped, bare frame only, approximate mass 8×10^5 kg
- ii with some bracing for cladding added
- iii with travelators almost complete
- iv with most of side cladding installed
- v with almost all cladding installed
- vi cladding complete

* not excited by preliminary walking/jumping tests

Table 2: Damping/% estimated from vibration free decay during construction

mode	name	measurement (see Table 1)			
		iii	iv	v	vi
1	LS1	-	0.34	0.38	0.36
2	VS1	0.16-0.21	0.18	0.33	0.335
3	VA1	-	-	0.48	
4	LA1	-	-	0.34	
5	TS1	0.165	0.25	0.38	0.38
8	VS2	-	0.55	0.40	

Table 3 Modal parameter estimates from low level vibrations during full-scale test

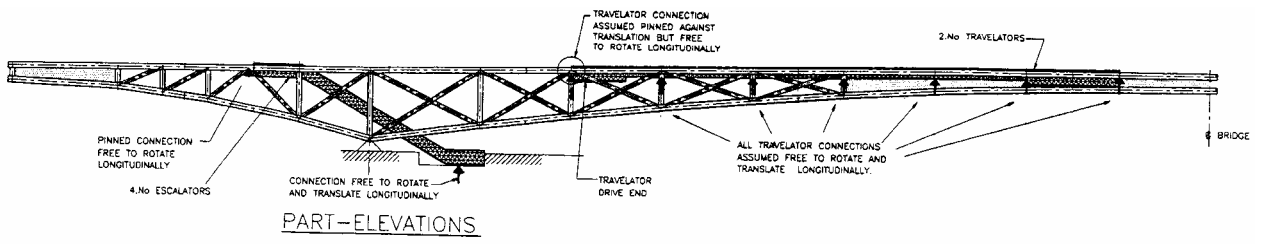
Mode r	Frequency f_r /Hz					Damping ζ_r /%			Modal mass m_r /1000kg		
	FRF	sine	jump	FEM ¹	FEM ²	FRF	sine	jump	FRF	sine	FEM ²
1/LS1	-		0.891	0.897	0.93	-	-	0.43	-	-	453
2/VS1	-	1.119	1.117	1.350	1.12	-	0.39	0.31	-	402	480
3/VA1	-	1.372	1.375	1.540	1.32	-	0.71	0.77	-	543	261
4/LA1	-	1.465	1.447	1.432	1.23	-	0.39	1.2	-	544	512
5/TS1	1.642	1.641	1.634	1.832	1.52	0.39	0.40	0.46	160	175	147
6/LS2	2.512	2.510	-	2.750		0.50	0.44	-	496		
7/TA1	2.703	2.701	2.687	3.007	2.05	0.39	0.30	0.75	317	337	213
8/VS2	-	2.751	-	3.063	2.30	-	0.37	-	-	134	312

1 FEM from LTA

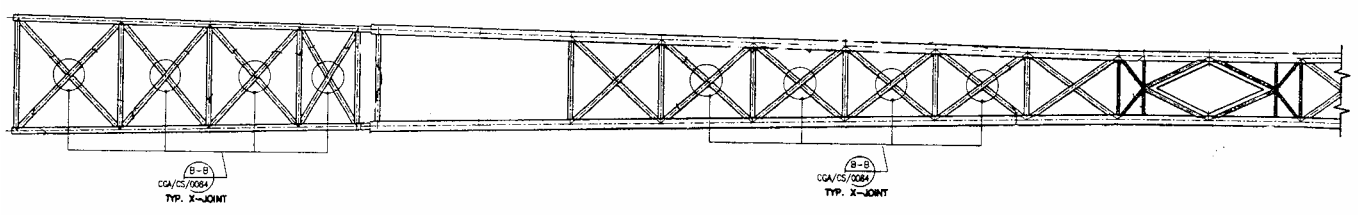
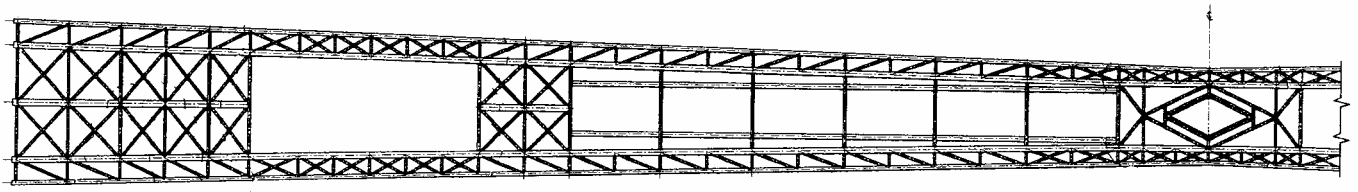
2 FEM from consultant report reference 11.

FIGURES

- Figure 1 Half elevation and sectional views of steel frame, showing measurements points BB...BP
- Figure 2 View from pin in March 2001, showing vertical framing system and cross-brace
- Figure 3 View of completed bridge with glass cladding from MRT platform level in January 2001
- Figure 4 Estimates of mode shapes and frequencies recovered from ambient vibrations in January 2001
- Figure 5 Top: build up and free decay due to single person swaying at 1.8Hz
close to bridge midspan
Middle: typical force signal for swaying at 1.8Hz, obtained from force plate
Bottom: response of 1000kg 0.9Hz oscillator to swaying force
- Figure 6 Spectrogram of lateral (upper) and vertical (lower) response
to workers walking across the bridge.
Light shading represents strong response, dominated by TS1 and LS1.
- Figure 7 Real part of driving point lateral (upper) and vertical (lower) inertance
frequency response function (FRF) from lateral chirp excitation.
Units of inertance are inverse mass, $(1000\text{kg})^{-1}$
- Figure 8 Lateral shaker force at BK with run up,
steady state response and rundown in lateral direction,
for mode LA1 at 1.47Hz
- Figure 9 Set of modes identified from free decay rundown from jumping and swaying
- Figure 10 Modes identified only from shaker testing
- Figure 11a Correspondence of TS1 mode shape from experiment and LTA FEM
- Figure 11b Correspondence of TS1 mode shape from experiment and LTA FEM
- Figure 12 Variation of damping and reduction of frequency with amplitude for modes VS1 and TS1



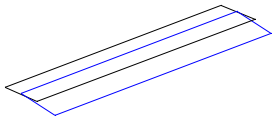
BB BD BF BH BK BM BP



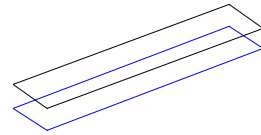




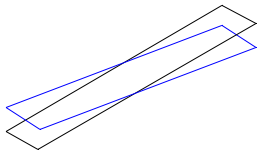
mode: 1 $f=0.9034\text{Hz}$



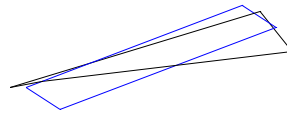
mode: 2 $f=1.135\text{Hz}$



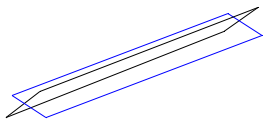
mode: 3 $f=1.386\text{Hz}$



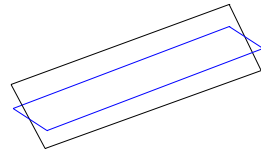
mode: 4 $f=1.476\text{Hz}$



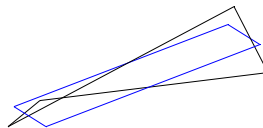
mode: 5 $f=1.655\text{Hz}$



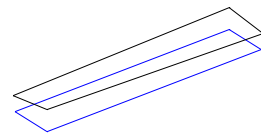
mode: 6 $f=2.505\text{Hz}$

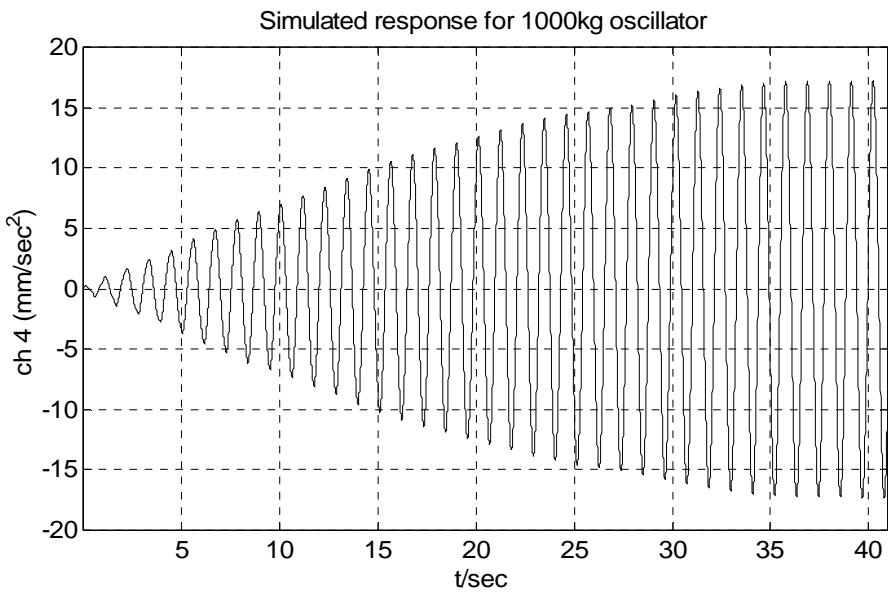
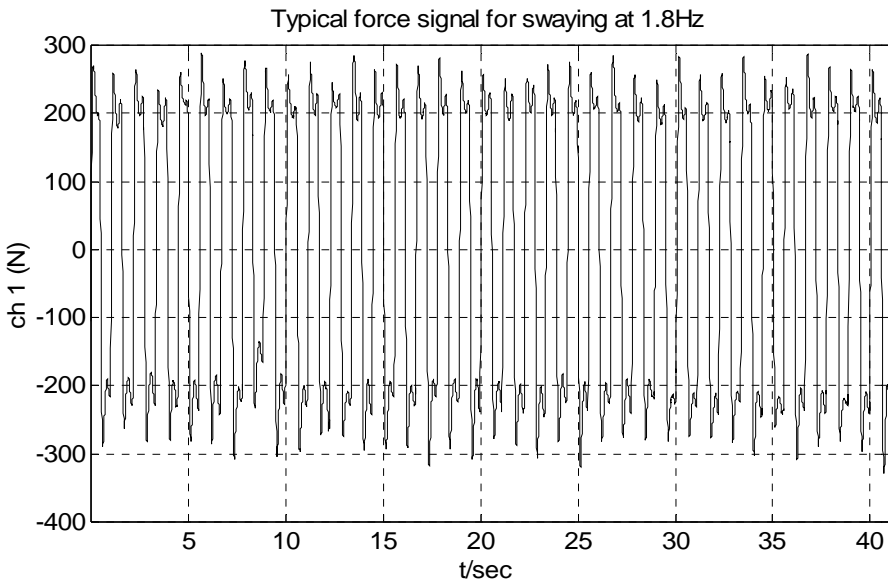
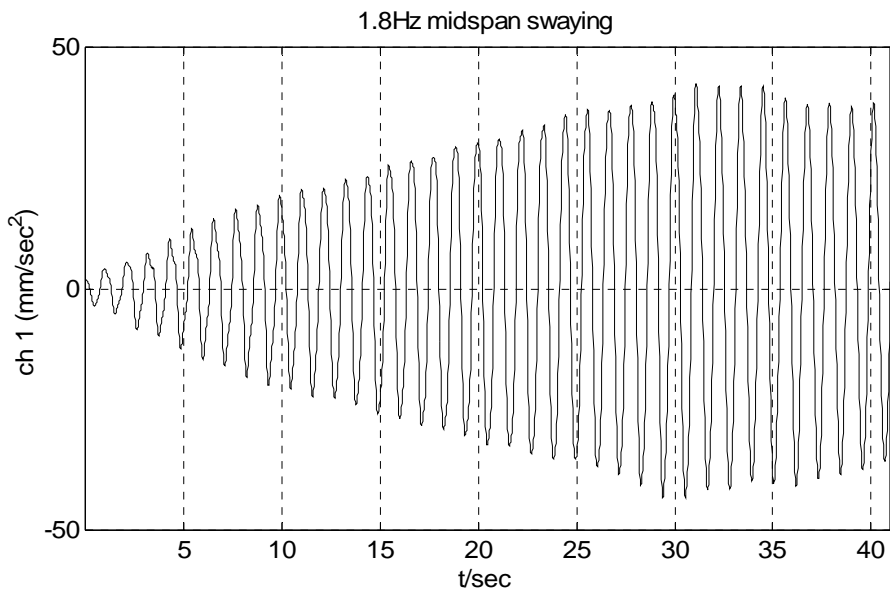


mode: 7 $f=2.692\text{Hz}$

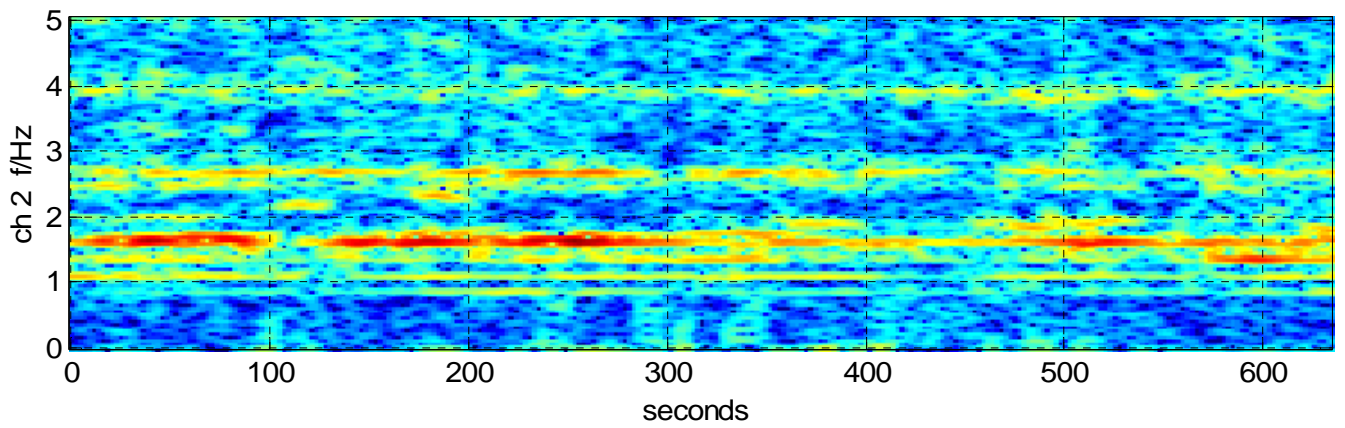
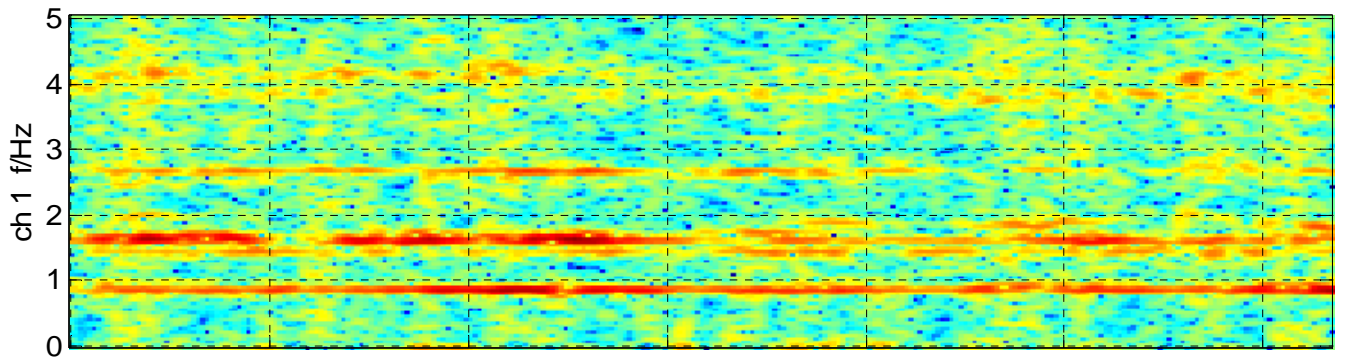


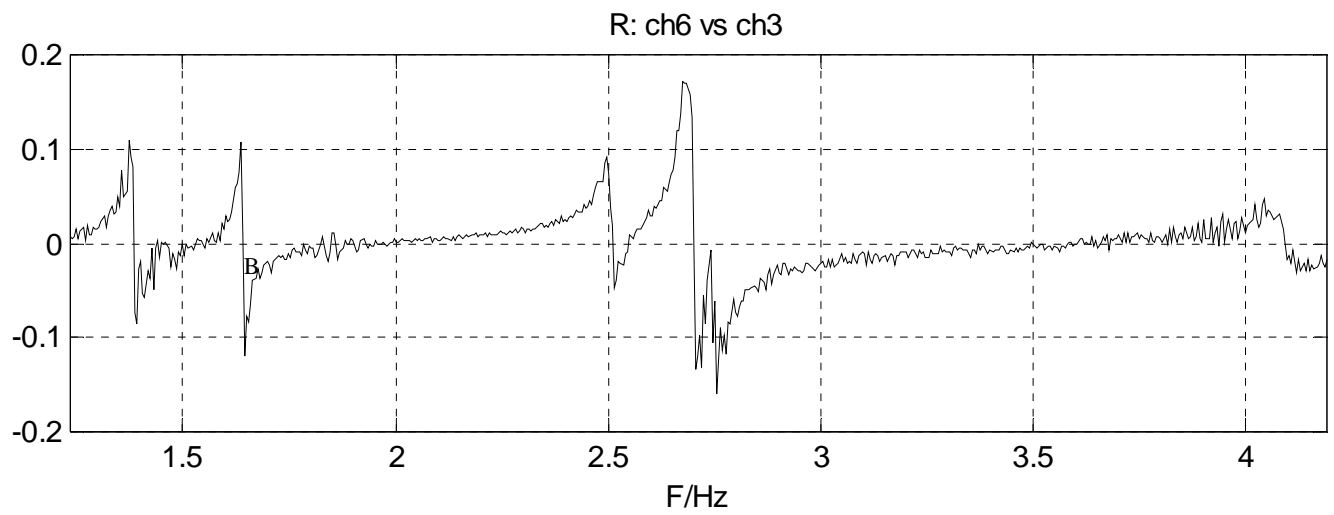
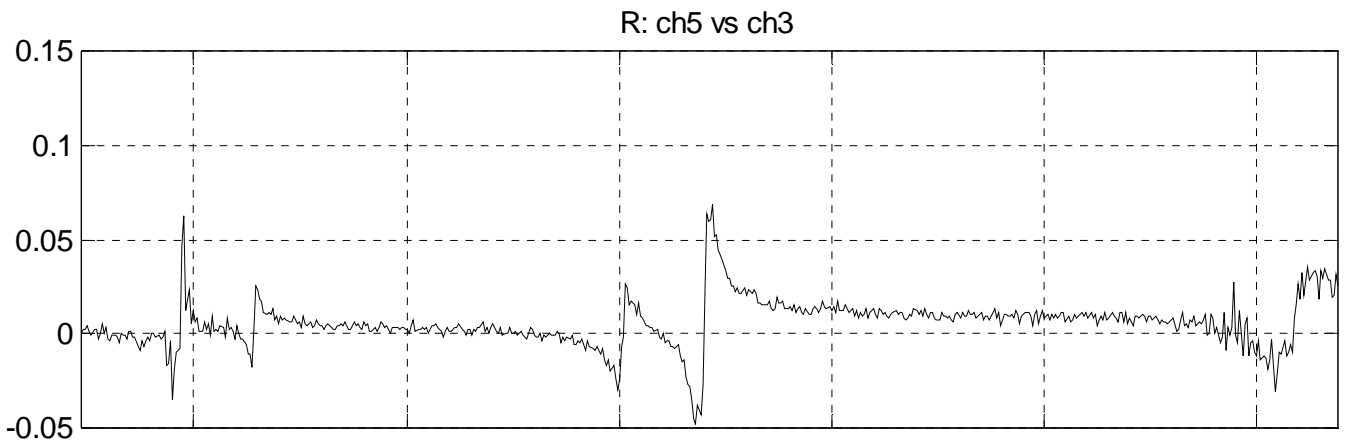
mode: 8 $f=2.736\text{Hz}$



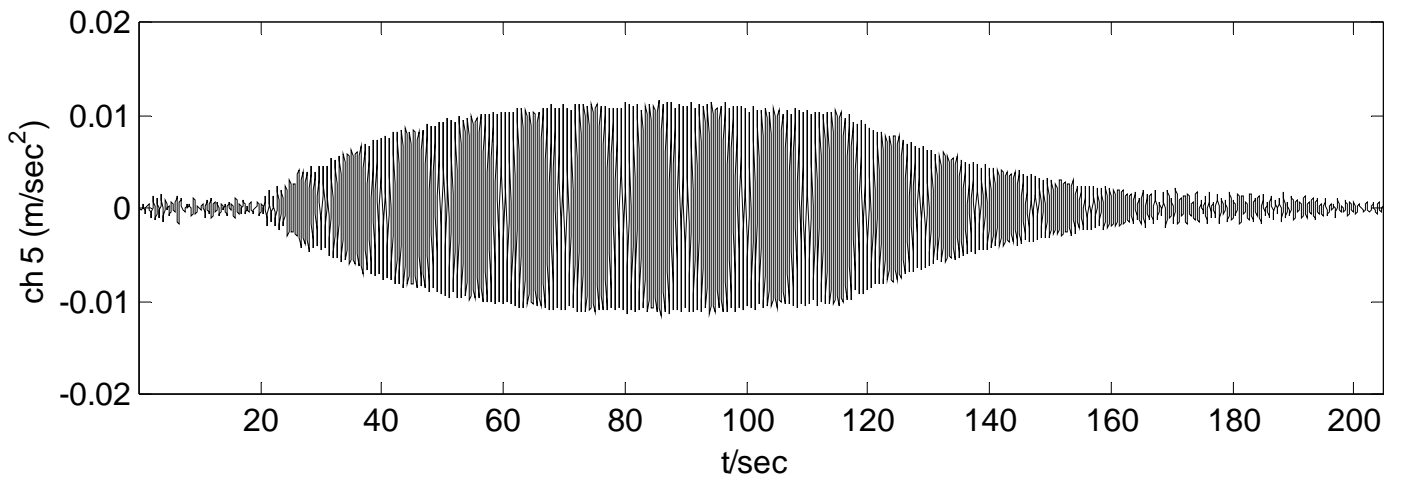
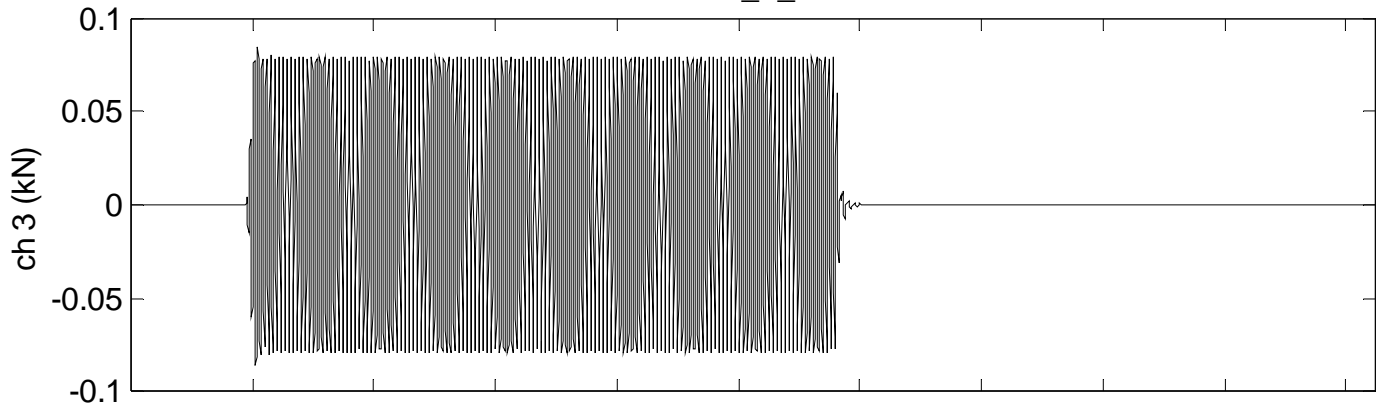


Jan14_7_amb

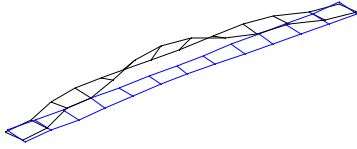




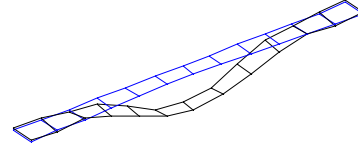
Test2h_1_147



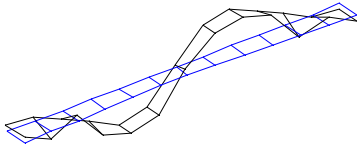
mode: 1 $f=0.891\text{Hz}$



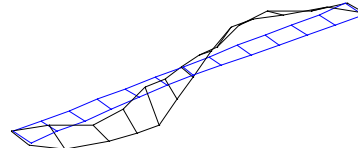
mode: 2 $f=1.117\text{Hz}$



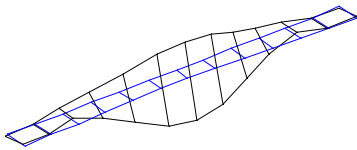
mode: 3 $f=1.375\text{Hz}$



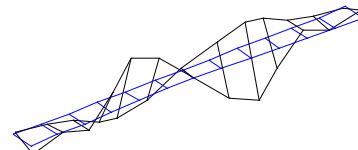
mode: 4 $f=1.447\text{Hz}$



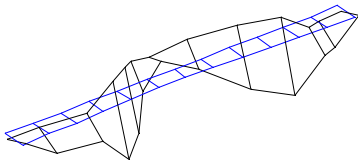
mode: 5 $f=1.634\text{Hz}$



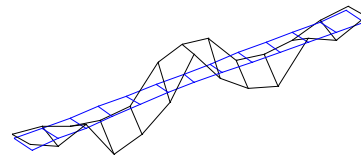
mode: 7 $f=2.687\text{Hz}$



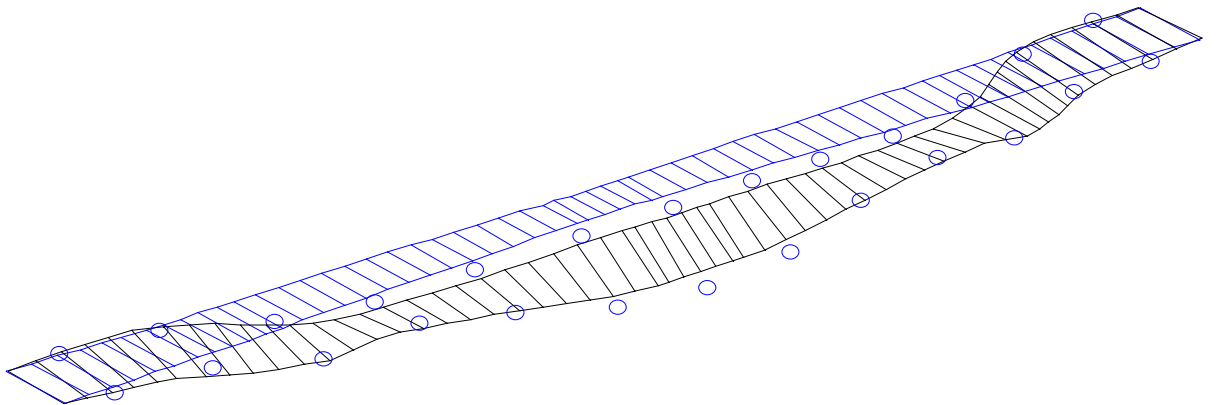
mode: 6 $f=2.51\text{Hz}$



mode: 8 $f=2.751\text{Hz}$



EMA: 1 f: 0.8911Hz, zeta: 0.43% FEA: 1 f: 0.9245Hz, MAC: 0.98201



EMA: 5 f: 1.6337Hz, zeta: 0.46% FEA: 5 f: 1.8563Hz, MAC: 0.99335

