

LONG SPAN STEEL PEDESTRIAN BRIDGE AT SINGAPORE CHANGI AIRPORT

PART 2: CROWD LOADING TESTS AND VIBRATION MITIGATION MEASURES

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SUMMARY

Following experimental and analytical studies of vibration serviceability of a 140m span steel footbridge which predicted excessive and uncomfortable vertical and lateral vibration levels due to crowd loading, a series of walking tests involving up to 150 pedestrians was aimed at assessing the prototype behaviour under ‘limiting typical’ pedestrian loads in two vibration modes judged to be critical.

In a walking test for possible instability resulting from so-called ‘synchronous lateral excitation’ (SLE), pedestrian volunteers were fed onto the bridge and told to walk casually. With all 150 available pedestrians circulating for several minutes, a steady increase in lateral vibrations was observed. This divergent response resembled the phenomenon observed during tests on the London Millennium Bridge (LMB), and while the maximum response reported here was an order of magnitude smaller than the largest levels reported for LMB on its opening day, it was distinctly uncomfortable for pedestrians. On the other hand, due to the lack of synchronization and random character of vertical loads together with enhanced damping due to the pedestrians themselves, vertical response levels were within acceptable comfort limits.

To mitigate the potential for strong and unsafe lateral oscillation in the unlikely event of larger numbers of pedestrians, a tuned mass damping system has been installed. The damping in LS1 has been increased by a factor of approximately four, so that SLE is effectively prevented for any foreseeable reasonable pedestrian loading.

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1 CHANGI MEZZANINE BRIDGE (CMB)

Changi Mezzanine Bridge (CMB) was opened in early 2002 and provides an underground walkway between two terminals at Singapore's Changi Airport. The bridge (1) is shown in its completed configuration in Figure 1, sitting inside a tunnel underneath an expressway between the two terminals, which also houses, at the lower level of the bridge, two platforms of a mass rapid transit (MRT) terminus.

According to the consultant, the bridge mass is 6,500 kg/m i.e. 1.3×10^6 kg total including all fixtures and fittings. The total deck area available for pedestrians, excluding voids and travelators is approximately 840m². Following the vibration serviceability problem with the London Millennium Bridge (LMB), finite element analyses and pedestrian-induced response predictions covering a range of loading scenarios were done for CMB by the consultant (2). At the suggestion of the consultant, Land Transport Authority of Singapore (LTA), who were project managers for the MRT terminus and all associated structures, commissioned full-scale modal testing of the completed structure (3) to validate the consultant's finite element model (FEM) and resulting predictions. The FEM was used for simulations of pedestrian loads that suggested possible serviceability problems with two particular vibration modes, and the modal testing showed a remarkably low level of damping, exacerbating these concerns. This paper describes an experimental program, effectively a proof test, to study the full-scale behaviour due to large numbers of pedestrians, to allay fears and to help define necessary mitigation solutions.

2 CROWD LOADING

In recent years there has been growing interest in the effects of crowds on flexible structures such as floors, cantilevered grandstands and footbridges, with a number of famous serviceability failures in footbridges (4) and grandstands (5) in which the structures were deemed unfit for the intended use. For footbridges the topic of vibration serviceability due to individual pedestrians is relatively well researched and codified (6). To a lesser extent, vertical loads from crowds have also been studied, for example Wheeler (7) studied response levels according to numbers of pedestrians crossing footbridges in Perth, Western Australia. As discussed by Bachmann and Amman (8), Wheeler's observations of amplitude dependence on square root of number of pedestrians N fit Matsumoto's model (9) that the effect of a single pedestrian is enhanced by the factor

$$m = \sqrt{\lambda T_o}$$

where λ is arrival rate of pedestrians and T_o is crossing time and λT_o is N , the number of persons on the bridge.

This square root dependence is logical if vertical forces from walking individuals are considered to be uncorrelated, and walking forces are described via power spectral density functions (10-12). Similar logic has been applied in the study of vertical response of LMB (13) where pacing rates have been assumed to be distributed normally.

Vertical walking forces due to a single pedestrian are usually presented in time domain in the form:

$$f(t) = W \left(1 + \sum_{n=1}^m G_n \cdot \sin(2\pi n/T + \phi_n) \right) \quad (1)$$

where the G_n are dynamic load factors (DLFs) on the static weight W , and the first harmonic i.e. the fundamental, at walking pace, is the strongest. Ebrahimpour (14) used a short section of instrumented walkway to derive values of DLFs dependent on numbers of pedestrians, for example for 'normal walking'

$$G_n = 0.25 - 0.2 \log_{10}(N) \quad (2)$$

with different values for different pacing rates. The frequency dependence of DLFs, particularly the fundamental G_1 , has been reported by the consultant (15).

Tests of crowd effects on floors have also been done in the UK (16) yet there appears to be no more reliable model for footbridge crowd loading than those described here. The comment by Wheeler (7) that the single pedestrian model as applied by British codes (17) may be the most appropriate deserves re-examination.

The above studies refer to vertical loads, and with the publicity surrounding LMB, recent interest has focused on lateral loads (13). The single most important result from these studies is that when pedestrians apparently synchronise their movements as the bridge sways, the oscillating lateral force for a single pedestrian is approximated by

$$G_{1,lateral} = 300V_{local} \quad (3)$$

where V_{local} is the peak lateral velocity where the pedestrian walks and the total load is proportional to number of pedestrians N .

Based on the above principles, the consultant for CMB conducted finite analyses and extensive simulations to predict response levels in lateral and vertical motion due to foreseeable numbers of pedestrians, such as in normal use or when fully occupied on special occasions. The predictions showed that two vibration modes would respond strongly to pedestrian crowd loading. The first lateral mode LS1 was predicted to occur close to 0.9Hz and so would respond to walking with footfalls at twice this frequency, while the first torsional mode TS1 predicted close to 1.6Hz would respond to footfalls at the same frequency.

A figure of 200 pedestrians (P_{200}) was believed to be a the maximum daily number of pedestrians likely to be occupying the bridge and the studies showed that vertical response levels due to this number of people would be unacceptable, at 3.6m/sec^2 exceeding comfort limits of $0.5\text{-}1.0\text{m/sec}^2$ (2). The analysis also demonstrated the possibility of synchronous lateral excitation with over 200 pedestrians, based on a-priori damping estimation of 0.7%, whereas the full-scale testing showed the figure to have a lower bound of 0.4%, suggesting a figure of less than 150 pedestrians for instability . Analytical predictions of mode shapes and frequencies were found to be close enough to the measured values, and modal masses were also reasonably close.

Having validated the frequency and mode shape predictions used in the numerical simulations, a full-scale pedestrian test program was planned to check the vibration serviceability predictions, involving measurements of vertical and lateral response during a sequence of crowd walking exercises.

3 PEDESTRIAN TEST PROGRAM

Sensors were located at strategic locations on the bridge to record response of the bridge during an afternoon of walking tests. Two sets of three accelerometers were arranged at approximately 1/3rd span ('BK') and mid span ('BP') locations. These are location BK and BP as shown in Figure 1 of reference (3). These were attached directly

to the top main chord members inside the cladding envelope, rather than to the deck surface, to avoid disturbance by pedestrians and local vibration of deck plates induced by pedestrians. For each location two accelerometers were aligned vertically, either side of the deck with a third mounted laterally so as to pick up vertical, torsional and lateral movement of the bridge. In addition, a LED-based optical system (18) was set up to measure lateral and vertical displacements either side of the deck at BP, giving a total of 10 active response channels. A video recorder was also used for study of pedestrian behaviour and for correlation of pedestrian numbers with response.

It proved surprisingly difficult to recruit a large number of volunteer pedestrians. At one stage there was a possibility to use national servicemen (conscripted soldiers), but in the end up to 150 pedestrians participated, mainly employees of LTA or of Changi Airport. Pedestrians were given written instructions and asked to walk casually, to not deliberately walk in step, and to move at their own comfortable pace. Consideration needed to be shown to comfort of volunteers and there was an intention to avoid very large response amplitudes. In other words this was not an attempt to reproduce the large amplitude oscillations observed at LMB.

A program of six tests was designed:

Test 1: Pedestrians were asked to walk to the end of the bridge main span and back on the opposite side.

Pedestrians were let on the bridge in groups of ten, spaced approximately 15 seconds apart, reaching a maximum of 150 pedestrians. After three minutes of circulation with the full set of 150 pedestrians, pedestrians were told to stop dead in their tracks and stand still; Figure 3 shows this condition. After observing the free vibration, walking was resumed, halted after a few minutes and then resumed to allow pedestrians to leave the bridge.

Test 2: As for Test 1 but with intervals of 30 seconds between groups and with walking only on one half-span. 130 of the pedestrians remained and all of these participated. Some pedestrians had apparently left because they were uncomfortable with the large (lateral) vibrations generated during the first test.

Test 3: In order to explore the passive damping capacity of pedestrians, this proceeded as for Test 2 but with approximately half the pedestrians (70) standing on the travelators close to mid-span.

Test 4: Travelators on one half span were set running in opposite directions. Pedestrians completed several circuits on these, at times walking, at times remaining stationary relative to the travelator movement. The test was in part aimed at identifying the vibration levels caused by operation of the travelators.

Test 5: Test 1 was repeated but with only 100 pedestrians (others had left) and using 30 second intervals between groups of ten.

Test 6: Four people jumped or swayed to a metronome beat to excite the first lateral, vertical and torsional vibration modes.

4 OBSERVATION FROM PEDESTRIAN TESTS

4.1 Test 1: up to 150 pedestrians circulating on the full bridge, with two dead stops

Figure 4 shows envelopes of mid-span lateral (L) and vertical (V) acceleration response on the main chord at midspan during Test 1. The signals are band-pass filtered to reveal the response in LS1 in the lateral acceleration and TS1 in the vertical acceleration. The superimposed stepped line indicates the number of pedestrians on the bridge who are walking, showing that both lateral and vertical vibration levels seem to increase with pedestrian numbers. It is also clear that after the maximum (150) is reached the lateral response continues to grow more or less steadily until the first dead stop (Figure 3) when a maximum peak to peak lateral acceleration in LS1 of 0.34m/sec^2 was recorded, corresponding to a displacement of 11mm peak to peak

For two segments with 82 second duration within the data of Figure 4, Figures 5 and 6 show the square root of auto-spectral density (ASD) of the raw unfiltered signals. Figure 5 corresponds to the period before 450 seconds when pedestrians are beginning to walk on the bridge, while Figure 6 shows the period around 700 seconds with the full set of pedestrians having maximum lateral response, just before the first halt to the walking.

Comparison of the upper plots in Figures 5 and 6 shows that for the lateral response (the peak below 0.9Hz) the amplitude has grown disproportionately to the increase in pedestrian numbers. The peak has shifted to a slightly lower frequency in Figure 6 and occupies a single spectral line, for the given frequency resolution of $(1/82)\text{Hz}$. In time domain, the response at peak amplitude is an almost perfect sinusoid with frequency 0.8775Hz.

Figure 7 shows time and frequency information in the same plot as a spectrogram. Within the limitations of time and frequency resolution, it is obvious that LS1 has steady levels with a very narrow frequency band while other modes grow and shrink while occupying a spread of frequencies.

Vertical response around TS1 is the same order of magnitude in the two periods; in both cases it occupies a rather broad band with similar root-mean square (RMS) response levels, around 20 mm/sec^2 . In fact the vertical response levels do not appear to change even while the lateral response is growing. A time domain system identification procedure (19) was used to extract modal parameters from the vertical response due to walking between first and second stops, before 1000 seconds, yielding frequency and damping estimates of 1.608Hz and 1% respectively for this part of the signal.

The relative 'sharpness' of the LS1 and TS1 peaks could be interpreted as the damping in LS1 reducing, with that for TS1 increasing. The alternate and currently popular explanation for growth of LS1 is lock in or synchronization of pedestrians walking at twice LS1 frequency in which case a clear and sharp non-resonant peak at two times LS1 frequency should appear in the vertical response.

Curve fitting of a decaying sinusoid to the strongest part of the lateral decay of LS1 in period 780-810 seconds when pedestrians are stationary shows the frequency to have risen a little to 0.884Hz but with 0.455% damping, almost the same as values observed on the unoccupied bridge for the largest response levels (Test 6). Response in TS1 dies out very fast but not with a smooth exponential decay, as pedestrians took several seconds to heed the instruction to stop walking. For the second full stop (after 1018 seconds) the torsional decay is more convincing, logarithmic decrement yielding frequency and damping estimates of 1.607Hz and 0.8%-1% damping, considerably different from values obtained for a range of amplitudes for the unoccupied bridge (Test 6).

4.2 Tests 2-5: variations on test 1 with fewer pedestrians

Even with no more than 100 pedestrians on the full span, the build up of lateral response without increase of pedestrian numbers is evident. In Test 2 a strong buildup was beginning to establish before pedestrians were told to stop, however similar levels could not be achieved even after resuming the walking and continuing for over four minutes.

Figure 8 shows the response during Test 5, with 100 pedestrians. Circumstances prevented a painstaking study of the minimum number of pedestrians for instability, but clearly 150 is enough and it seems that for a smaller number of pedestrians, it may be necessary to wait longer for the instability to appear. The vertical response in TS1 always built up quickly when pedestrians started to move and decayed very fast when walking ceased. This is apparently the result of extra damping of the stationary humans on the structure (14), discussed later.

Comparison of Test 2 and Test 3 data do not reveal any obvious changes to damping ratio during movement of the maximum number of pedestrians, while the operation of the travelators generated a low level of vibration at relatively high frequency, 11.5Hz, together with a little excitation of the low frequency bridge modes.

4.3 Test 6: deliberate jumping and swaying

Figure 9 shows the effect of four people swaying and jumping at mid-span. Despite best efforts, the lateral response levels of Test 1 could not be achieved, although vertical response in mode TS1 was stronger than achieved in the walking tests. At the largest amplitudes (values in brackets) damping and frequency estimates are:

LS1:	0.41%	0.886Hz	(+/-0.08 m/sec ²)
VS1:	0.42%	1.112Hz	(+/-0.35 m/sec ²)
TS1:	0.59%	1.616Hz	(+/-0.38 m/sec ²)

As reported in the companion paper (3), both mode TS1 and VS1 frequencies have strong amplitude-dependence, with frequency increasing and damping reducing as amplitudes diminish.

For LS1 the values of frequency and damping were little different from those of the structure when occupied by 150 pedestrians. In other words for lateral response, standing humans have negligible effect on the system dynamics. On the other hand TS1 parameters are strongly affected by occupancy as well as by response levels.

5 ESTIMATION OF FORCES DURING PEDESTRIAN TESTING

The procedure reported by the consultant for LMB (13) has been applied to the response measurements for Test 6 and then Test 1 to estimate the harmonic forces generated by the pedestrians. By considering the bridge as a single degree of freedom system, the amplitude of net applied modal forces f_e is given by:

$$f_e = 2\zeta_r m_r \hat{y}_r + m_r \Delta \hat{y}_r / \pi \quad (4)$$

where \hat{y}_r , $\Delta \hat{y}_r$ are respectively amplitude of modal acceleration and change in amplitude of modal acceleration over one cycle. Values of modal mass m_r and modal damping ζ_r were taken from the full-scale testing (3). The first term in Equation (4) represents a force to overcome damping, the second is the 'inertia force' to increase amplitude. Modal masses are unity-normalised, and as the response is measured at the point of maximum response, the modal accelerations correspond to the physical accelerations.

The clearer responses generated during test 6 were first examined to test the procedure. Figure 10 shows the components of Equation 4 for the build-up and decay of mode LS1 response (from 250-350 seconds in Figure 9). The net force is the input from the four test subjects, which is seen to achieve a steady value of about 1kN for long enough to build up significant response. Hence one person can generate a force with approximate amplitude

250N by swaying, a figure confirmed by laboratory measurements using a force plate (3). Despite the heavy low pass filtering, oscillations appear in the net force (which should be zero) as the inertia force depends on a response which ‘beats’ at a very low frequency.

The effect for VS1 (380-500 seconds in Figure 9) is shown in Figure 11. It is clearer than LS1 and shows that each jumper generates forces with 1kN amplitude at VS1 frequency. For TS1 (Figure 12) there was a false start in the jumping and the forcing function per person is (surprisingly) smaller than for VS1.

Given that the above results show that the procedure produces believable estimates of input force, Figure 13 applies the same analysis to data from Test 1. Two notable periods are 750-850 seconds and 980-1050 seconds i.e. before and after the two stops when the vibrations had just peaked. In the first case the modal input force peaked at 0.9kN before the dead stop, corresponding to a modal force amplitude per person of 6N. At this level the velocity (amplitude) was 32mm/sec, hence the constant of proportionality for equation 3 is found to be 188Nsec/m without accounting for mode shape, approximately double that if mode shape effect on velocity and force are accounted for. For these calculations the modal mass was taken as 4.53×10^5 kg and the modal damping as 0.43%, values obtained from the preliminary vibration testing (3). Note that during the periods when walking had ceased the net force averaged to zero, showing the cancellation of damping and inertia forces.

Bachmann (20) reports that for walking experiments with LMB, 30-50% of pedestrians should be synchronised for a lateral displacement amplitude of 5mm. He also provides a value of 23N for the fundamental component of lateral walking force for a 584N person, a value supported by our own studies. Although there was no observable synchronisation of the pedestrians, 0.9kN of correlated force would require approximately 40 pedestrians at midspan, already 26% of the total, more if the mode shape scaling of distributed pedestrians is accounted for. We did not observe any such large scale synchronisation of pedestrians.

6 VERTICAL RESPONSE AND CROWD EFFECTS

For vertical response in TS1 measured at the midspan chord, the response shows relatively correlation with pedestrian numbers that goes approximately with the square of the number of pedestrians (12). The strongest response was achieved after the first dead stop; the pedestrians started to walk again almost in unison, resulting in a rapid build up of response.

The same levels of response (around 100mm/sec^2 amplitude) were achieved with only three pedestrians walking in deliberate synchronization to a prompt (metronome) at 1.64Hz . Figure 14 shows the analysis; from 0 to 50 seconds the pedestrians walked from mid-span to the span end, then returned on the opposite side. Response built up rapidly but also decayed when the walking became desynchronised with the vibrations, and the input force was working to reduce vibration levels. The pedestrians continued to the other end of the span and return, coming to a dead stop at 210 seconds back at midspan, at which point the maximum response amplitude of 96mm/sec^2 was registered. The peak input force amplitude is shown as 350N and with perfect timing larger response could have been achieved.

Figure 14 also shows the envelope of response that would be achieved using the BS5400 (perfect) pedestrian load of 180N moving at 1.47m/sec . The response has been calculated by applying the load at exactly TS1 frequency, modulated by the measured mode shape, and using modal mass and damping values of $1.47 \times 10^5\text{kg}$ and 0.46% from the vibration test (3). Peak response amplitudes for the BS5400 simulation and the three-pedestrian experiment exceed the largest response generated by 150 pedestrians as shown in Figure 5, and it is for this reason that Wheeler (1) stated an opinion that the single (test) pedestrian is the most appropriate excitation model.

7 MODAL PARAMETERS DURING WALKING TEST 1

There is evidence that if the walking forces are viewed as a random process with a power spectral density function that varies more smoothly than the sharp modal frequency response functions, then the damping factors observed during walking tests were significantly different from values for the empty structure.

For mode LS1, damping appears to reduce during walking but not when pedestrians are stationary, while for TS1 it appears to increase during walking, but is little different when pedestrians stop moving. For mode LS1, frequency changes only very slightly while for TS1 there is a significant drop. For LS1 the apparent reduction in damping has already been ascribed to apparent synchronization of pedestrians as a 'correlated lateral force'.

A plausible explanation for the effect on TS1 is the human-structure interaction mechanism (20,21) in which a single human is modelled as a spring-mass-damper system, so that the combined human-structure system is considered as a two degree of freedom (2DOF) linear dynamic system. Taking values of $k=2\text{kN/m}$, $m=80\text{kg}$ and $c=80\text{Ns/m}$ for a human, $k=17\text{MN/m}$, $m=1.47 \times 10^5\text{kg}$, $c=13\text{kNs/m}$ for the bridge and assuming the humans are all at the point of maximum response leads to a damping increased to 1% , consistent with the observations. However in the 2DOF model the frequency should also increase, to 1.66Hz , an effect that was not observed in any of the tests. Instead the observed frequency was, at 1.61Hz , lower than the value 1.616Hz achieved with much larger response

amplitudes. Taken as a distributed rigid mass, 150 pedestrians would cause a 0.5% drop in frequency, to 1.608Hz, close to that observed.

8 DISCUSSION

Adopting the concept of 'lock-in' leading to synchronous lateral excitation, it can be argued on the basis of the above observations that 150 pedestrians exceeds the 'critical' number needed to produce a net force exceeding the structural damping force i.e. to produce 'negative damping'. The lock-in that is a prerequisite seems to be slow to establish and the resulting growth is also relatively slow. Compared to one famous test of LMB, the growth was much slower, possible due to the larger mass (CMB is three times heavier for comparable sub-structures).

In the 150-pedestrian walking test (Test 1), lateral acceleration levels at mid-span in mode LS1 approached +/- 0.17m/sec² due to lateral excitation. The lateral response levels increased more or less steadily as more pedestrians moved onto the bridge, but even with a constant number of pedestrians there was a period of three minutes during which mode LS1 response continued to grow steadily. At a point where the levels seemed to have become uncomfortable the walking was halted.

From watching video recordings of the testing there is no indication that pedestrians were synchronised to any visible degree, neither is there clear evidence from examining vertical response at twice LS1 frequency. If 40 synchronised pedestrians each generate a vertical load at twice the frequency of mode LS1 having a realistic amplitude of 150N, a sharp non-resonant response peak with RMS up to 0.12m/sec² would be seen in vertical response (Figure 6), assuming all pedestrians are on one side of the deck. Close examination of vertical response shows that RMS response in the range 1.7Hz to 1.8Hz never exceeded 0.01m/sec², corresponding to an imbalance of three or four synchronised pedestrians between each side of the deck. Even so, there is at present no better explanation for the lateral response levels than synchronisation, a lateral force proportional to structure velocity and a constant of proportionality similar to that found at LMB.

While the growth in lateral response was not dramatic it was clear, and it is certain that with larger numbers of pedestrians, the response would grow more readily. The consultant's figure of 0.4m/sec² for LS1 response at P₂₀₀ is believable, indeed the same figure could have been achieved had the P₁₅₀ response been allowed to continue to grow.

In any case a significant number of pedestrians felt sufficiently uncomfortable after Test 1 that they did not want to participate in further testing. A small number of comments were solicited and it seems that the discomfort was most severe when the pedestrians had stopped walking.

Torsional response levels at mid-span in vertical direction at the main chord were much smaller than predicted by the consultant, and were similar to maximum levels predicted for a single pedestrian with perfect timing. Other modes such as first vertical mode VS1 were only weakly excited.

Jumping and swaying in Test 6 showed that with some determined effort, accelerations up to 0.4m/sec^2 amplitude in modes VS1 and TS1 could be generated by only four people. By comparison, vibrations in LS1 were hard to establish to even 50% of the Test 1 values with deliberate effort.

Apparent increases in damping and drop in frequency for TS1 are only partially explained by a proposed model of humans as mass-spring-damper systems, but it is clear that for vertical response, added damping can be assumed for both stationary and walking humans. For lateral excitation there is no such beneficial effect and even a weak non-linearity of structural damping will not mitigate the lateral forcing by pedestrians.

9 SOLUTION: TUNED MASS DAMPER

The pedestrian testing has shown that, excepting the situation of informed and deliberate forcing of torsional response, it is the lateral response which could reach uncomfortable and unstable levels of vibration. Whereas for LMB it was possible to increase lateral damping by making use of relative movement across diagonals, the rigid framing system of CMB allows no such possibility. Hence the natural solution for response mitigation was a tuned mass damping (TMD) system optimized to control mode LS1.

The TMD system comprises a pair of dampers installed within the skeleton frame either side of the mid-span opening. Each unit (Figure 15) has a pendular mass of 500kg, a natural frequency of 0.886Hz and a viscous damping coefficient of 150Ns/m. To test the functioning of the TMD system, two people swayed to up to 20 beats of a metronome set to 107 beats per minute, in order to induce the maximum possible sway before standing perfectly still and observing the free decay. In addition, direct forcing of the TMD mass was used by the contractor, pushing the mass while reacting against the frame of the bridge to generate a transient response.

The TMD function testing was conducted 18 months after the bridge was opened and the mode LS1 parameters were rechecked with the TMD inactive. For amplitude of 0.05 m/sec^2 , the measured frequency and damping were

0.89Hz and 0.45%. Assuming the bridge behaves as a SDOF system in mode LS1, the parameters of the bridge and TMD which are respectively

	mass	stiffness	damping constant
Bridge mode LS1	453,000kg	14.17MN/m	22.85kNs/m
TMD (each):	500kg	15.39kN/m	150Ns/m

lead to the following set of modal parameters for the two-degree of freedom system:

mode	frequency	damping	TMD amplitude and phase w.r.t LS1
1	0.8681Hz	1.72%	25.6, 152°
2	0.9052Hz	1.44%	17.54, -151°

Figure 16 shows the test result; compared with a maximum achievable amplitude of 0.1m/sec² with the TMDs inactive, the top plot shows that only 0.05m/sec² could be generated when both were released. The damping measured with the TMD active was no less than 1.65%, corresponding to an increase by a factor close to four. The lower plot of Figure 16 shows the acceleration of one TMD mass, which is between 3 and 5 times larger than bridge and lags it by 38°. The frequency of the free decay varies from 0.896Hz to 0.906Hz as the amplitude decreases. While the response and effect of the TMD does not quite match the theory, the required increase in damping has been delivered and the critical number of pedestrians for synchronous lateral acceleration can now be revised to a minimum of 560. Judging by the density of pedestrians during the test and the operational conditions this level of crowding is extremely unlikely.

10 TIMELINE FOR CHANGI MEZZANINE BRIDGE

Design awarded to ARUP (New York) and Skidmore Owing & Merrill	4 th July 1997
Design finalized	February/March 1999
London Millenium Bridge opened	10 th June 2000
London Millennium Bridge closed	12 th June 2000
Construction begins	June 2000
Report on vibration serviceability issued	December 2000
Walking tests to validate ARUP theory on SLE	19 th December 2000
Skeleton frame complete and un-propped	June 2001
Prototype test	31 st January 2002
Walking tests	1 st February 2002
First train to Changi MRT station (bridge open to public)	8 th February 2002
TMD installed and tested	July 2003

11 CONCLUSIONS

Before the crowd test with up to 150 pedestrians, based on the modal testing and analyses by the consultant, two vibration modes were considered to be have potential vibration serviceability problems. Lateral mode LS1 with a modal mass of 4.53×10^5 kg, a frequency close to 0.9Hz and damping of up to 0.45% could be excited to a level of discomfort by a small number of dedicated swaying volunteers. Concerted effort by one or two perfectly timed jumpers could excite mode TS1, close to 1.64Hz, to visible vertical motion due to the low modal mass and damping.

To investigate the possibility of excessive vibration in either mode during normal pedestrian use, a proof test was organized using up to 150 pedestrians walking normally over the main span of the bridge. The exercise demonstrated that vibrations in mode TS1 were well controlled, with enhanced damping due to the pedestrians themselves, whether walking (casually) or standing perfectly still. On the other hand the vibration levels in LS1 began to grow in a manner consistent with synchronous lateral excitation (SLE). The number of pedestrians causing this was consistent with the prediction using the low damping estimate obtained from the modal testing.

Detailed examination of response signals showed that for mode LS1 either very low damping or almost pure harmonic excitation resulted from the presence of the pedestrians and that a 'correlated modal force amplitude per pedestrian' up to 6N was achieved, although there was little direct evidence of any correlation between pedestrians. Neither was there any sign of correlated forces exciting non-resonant response in the vertical response.

Whatever the mechanism, to prevent future occurrence of SLE, a tuned mass damper (TMD) system has been installed, which has increased mode LS1 damping by a factor close to four, with a corresponding increase in the number of pedestrians required to induce SLE. As it is highly unlikely to ever experience such crowd numbers, the conclusion is that the bridge is now serviceable under all reasonable pedestrian loading conditions.

Despite predictions of possible excessive response levels in vertical vibration with a large crowd of pedestrians, measured response levels in the critical mode TS1 were no greater than the level predicted using the British Standard 'perfect pedestrian' walking at exactly the mode frequency. This adds to the body of evidence suggesting the need for better guidance on modeling vertical loading from pedestrian crowds.

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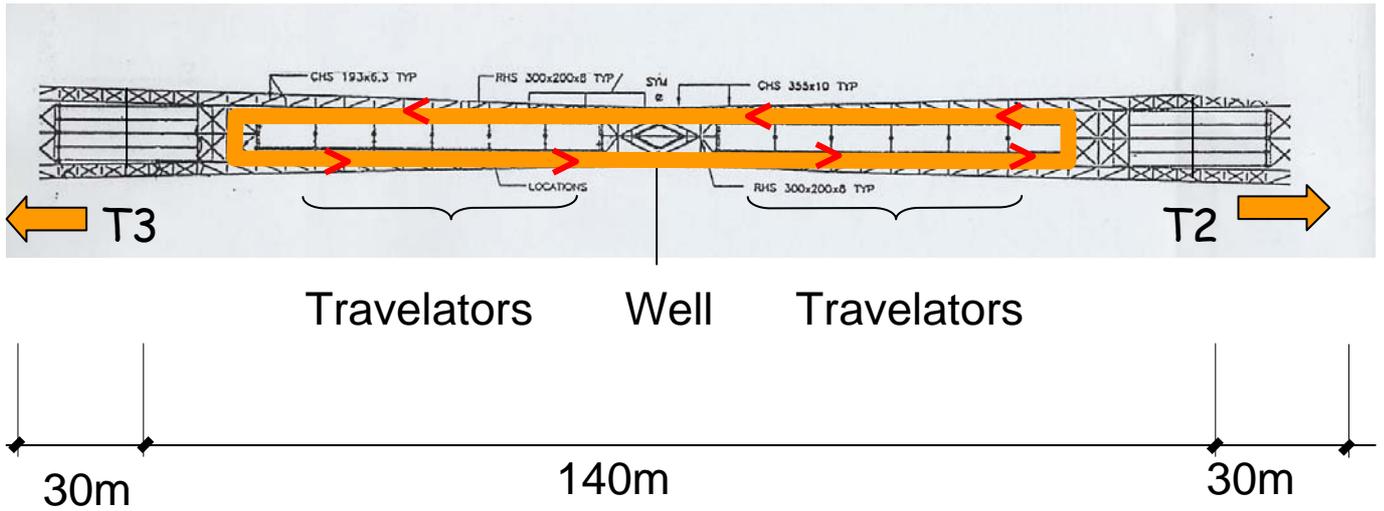
REFERENCES

- 1 Roche M, 'Performance in lieu of prescription in bridge design – Changi Airport mezzanine bridge, Singapore' *Transportation Research Record* **1770**, 2001, 195-203.
- 2 Roche M, Land Transport Authority Changi Bridge Vibration Characteristics, Ove Arup & Partners USA, December 2000
- 3 Brownjohn JMW, Fok P, Roche M, Moyo P, Long span steel footbridge at Changi Airport. Part 1: Prediction of vibration serviceability problems
- 4 Beard M, Why did the bridge wobble so much? $F=k \times v$. Independent, 21 Feb 2001
- 5 Wilkinson ER, Coombe JP, 'University of Illinois Memorial Stadium: Investigation and rehabilitation', *ASCE Journal of Performance of Constructed Facilities*, **5**(1), 1991, 2-15.
- 6 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.
- 7 Wheeler JE, 'Prediction and control of pedestrian-induced vibration in footbridges', *ASCE Journal of Structural Engineering* **108**(9) 1982, 2045-2065.
- 8 Bachmann H, Ammann W, *Vibrations in structures induced by man and machine*. Structural Engineering Documents 3e, IABSE, Switzerland, 1987.
- 9 Matsumoto Y, Nishioka T, Shiojiri H, Mastsuzaki K, 'Dynamic testing of footbridges', IABSE Proceedings, P-17/78, August 1978.
- 10 Mouring SE, Ellingwood BR, 'Guidelines to minimize floor vibrations from building occupants' *ASCE Journal of Structural Engineering* **120**(2) 1994, 507-526.
- 11 Eriksson P-E, *Vibration of low-frequency floors – Dynamic forces and response prediction*. Doctoral Thesis D 94:3, Chalmers University, 1994.
- 12 Brownjohn J M W, Pavic A, Omenzetter P, A spectral density approach for modelling continuous vertical forces on pedestrian structures due to walking. *Canadian Journal of Civil Engineering* (in press)
- 13 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.
- 14 Ebrahimpour A, Hamam A, Sack RL, Patten WN, 'Measuring and modelling dynamic loads imposed by moving crowds', *ASCE Journal of Structural Engineering* **122**(12) 1996, 1468-1474.
- 15 Young, P. 2001. Improved floor vibration prediction methodologies. Proceedings of Arup Vibration Seminar on Engineering for Structural Vibration – Current Developments in Research and Practice. IMechE, London.
- 16 Ellis BR, 'On the response of long-span floors to walking loads generated by individuals and crowd', *The Structural Engineer* **78**(10) 2000, 17-25.
- 17 BSI 2001. Steel, concrete and composite bridges. Specification for loads, BS5400: Part 2 (BD37/01, Appendix B). British Standards Institution. Milton Keynes, UK.
- 18 Tervaskanto M, Ahola R, 'A position-sensing laser system for analyzing dynamic deflection in large constructions' *Field measurements in Geotechnics*, 1991, 129-137, Balkema.
- 19 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.
- 20 Brownjohn JMW, 'Energy dissipation from vibrating slabs due to human-structure interaction' *Journal of shock and vibration* **8**(6), 2001, 315-323.
- 21 Sachse R, Pavic A, Reynolds P, 'Human-structure dynamic interaction in civil engineering dynamics: a literature review', *The Shock and Vibration digest* **35**(1) 2003, 3-18.

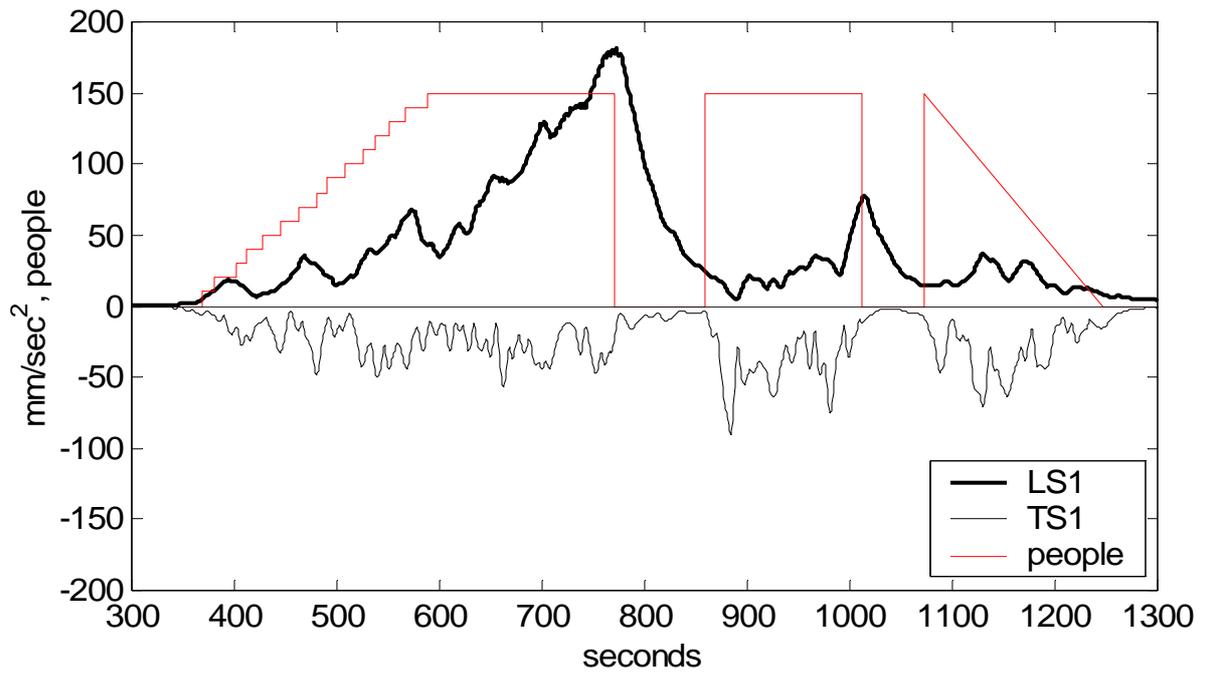
FIGURES

- Figure 1 Changi Mezzanine Bridge viewed from mezzanine level
- Figure 2 Schematic for walking Test 1
- Figure 3 Test 1 with 150 pedestrians standing still during decay of strong lateral vibrations
- Figure 4 Envelopes of band-pass filtered mid-span lateral and vertical acceleration response during Test 1: up to 150 pedestrians circulating round full length of main span. Positive half shows is lateral response around LS1 frequency, Negative half shows vertical response around TS1 frequency.
- Figure 5 Fourier amplitudes of mid-span lateral (upper) and vertical (lower) response during Test 1 in the period 380-450 seconds, with pedestrians joining and numbers increasing
- Figure 6 Fourier amplitudes of mid-span lateral (upper) and vertical (lower) response during Test 1 in the 655-735 seconds with the full set of pedestrians. LS1 is (almost) a single spectral line at this resolution.
- Figure 7 Time-frequency plot (spectrogram) of lateral (ch 5) and vertical (ch 6) response during Test 1. Intensity of colour (strongest is black, weakest is blue) shows frequency, bandwidth and timescale of various modes.
- Figure 8 Envelopes of band-pass filtered mid-span lateral and vertical acceleration response during Test 5: up to 150 pedestrians circulating round full length of main span. Positive half shows is lateral response around LS1 frequency, Negative half shows vertical response around TS1 frequency.
- Figure 9 Test 6: Mid-span response to four people swaying to excite LS1 then jumping to excite VS1 and TS1
- Figure 10 LS1 modal forces during Test 6
- Figure 11 VS1 modal forces during Test 6
- Figure 12 TS1 modal forces during Test 6
- Figure 13 LS1 modal forces during Test1
- Figure 14 Mid-span vertical acceleration amplitudes due to three people completing one circuit of the bridge with footfalls timed at 1.64Hz, and comparison with BS5400 simulation
- Figure 15 Tuned mass damper with 500kg pendular mass fixed on skeleton frame
- Figure 16 Performance of dual tuned mass damper (TMD) system. Upper plot (ch1) shows build up and free decay in mode LS1, lower plot (ch3) shows movement of TMD mass.

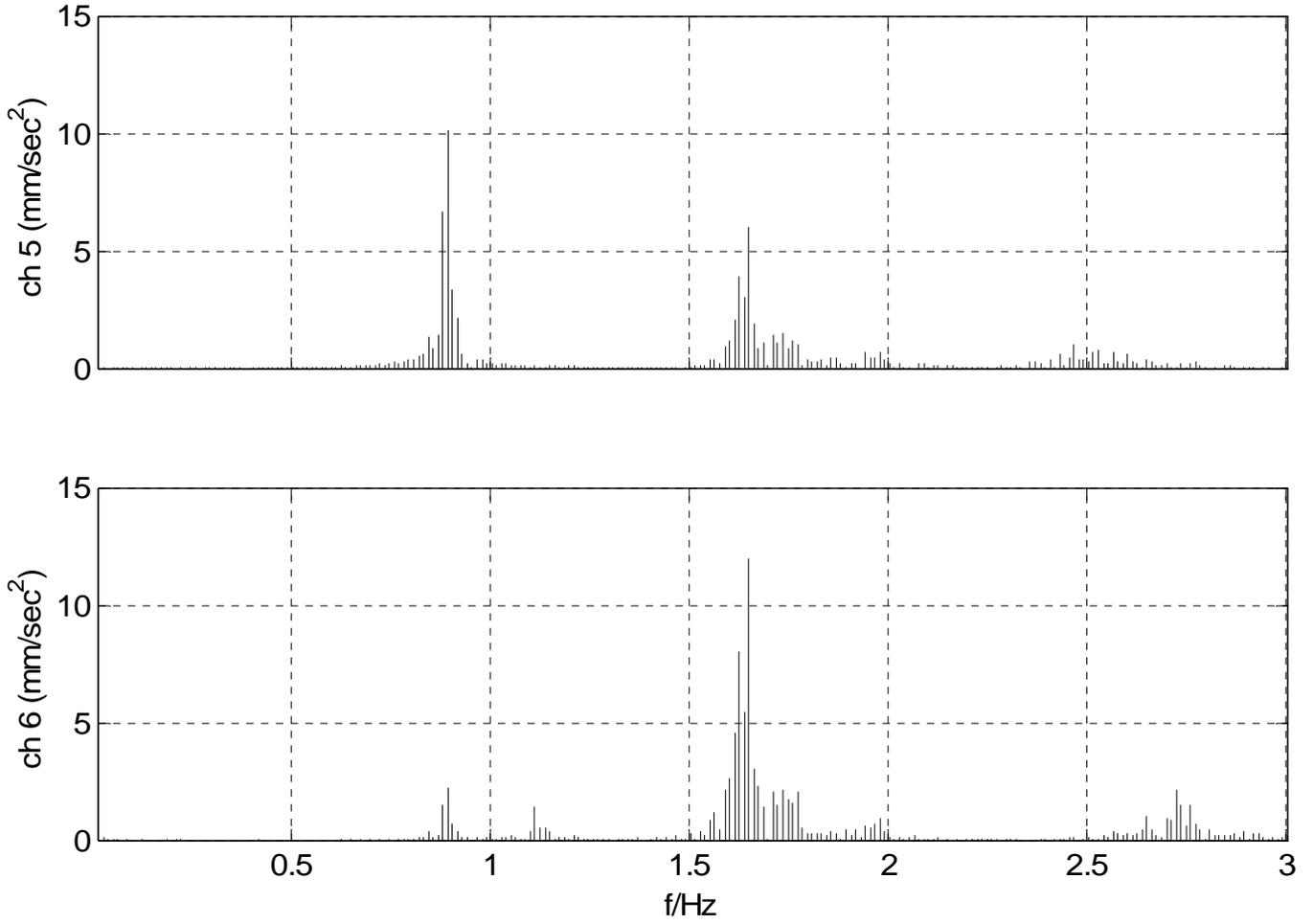








Test 1



Test 1

