Assessment of the dynamic characteristics of the Helix Bridge at Marina Bay, Singapore

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Introduction

The Helix is a 280m pedestrian bridge in Singapore connecting the promenade at Bayfront and the Marina promenade. It forms part of a 3.5km waterfront promenade that loops around Marina Bay. Built using duplex stainless steel, the bridge has a unique curved helical geometry which consists of two intertwining helices that spiral in opposite directions (Figure 1). It was designed by a team comprising global consulting engineers Arup, and architects from the Australian Cox Group and Singapore-based Architects 61.

Due to its location in the centre of Marina Bay, Singapore's new downtown area, there are many possible usage scenarios for the Helix in addition to its primary function of providing pedestrian access. These include: a crowd gathering on one side of the bridge to view a firework display; and the bridge acting as a sports or events venue, e.g. for marathons or performances. The complexity of possible loading scenarios also extends to the accidental impact force from vessels in the bay. The Helix's structurally efficient long-span form, coupled with its long cantilever viewing pods and the possible functioning scenarios, meant that the structure's dynamics and vibration performance were a major design consideration to ensure user comfort.

The inception of a vibration study required a detailed review of international standards and literature on the vibration performance of pedestrian bridges. The bridge's dynamic properties were first analysed numerically with computer simulation, and subsequently validated with physical tests on site. The measured dynamic properties and response generated compare favourably with various published acceptability criteria. This paper describes the tests performed and results achieved.

Structure

The bridge is made up of two end spans of 40m each and three interior spans of 65m each. Cantilevered pods of approx. 9m cantilever spans are extended from bridge support locations. The bridge is founded on bored piles. The structural system comprises two sets of separate spiralling helices that encircle the entire length of the bridge, meeting only at the deck level. Each set of the spiralling steel members is held together by a series of light struts, forming the loop stiffening rings or loop frame (Figure 2). Together, these light struts form a helix and, along with the tension rods that hold the two helices together, provide the stiffness

Figure 1 Structural form of Helix Bridge resembles DNA structure

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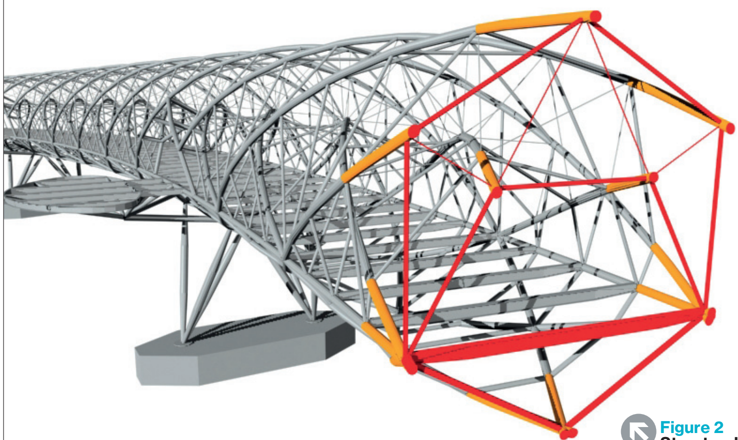

Figure 2
Structural geometry of bridge

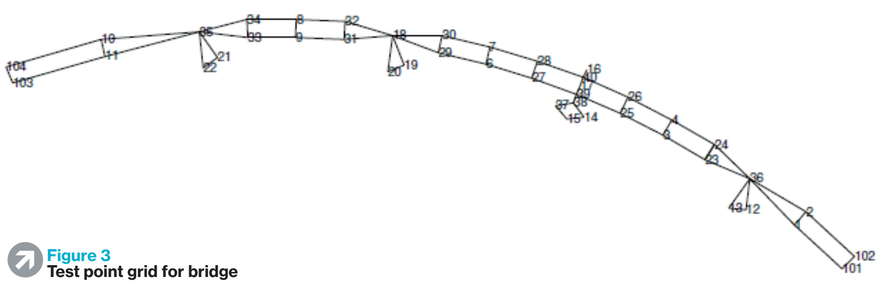

Figure 3
Test point grid for bridge

to the bridge at 2.7m intervals along the bridge form. The loop frame also provides buckling restraint to the spiral steel members.

A spiral or helix form is structurally inadequate. It will not span between two supports to carry load and the helix mechanism will untwist when it is loaded. However, two helices can work together to form a bridge when they are run in opposite directions and interconnected by a series of struts and tie rods. As they try to untwist, the rods work in two ways – they connect the two spirals together and, at the same time, keep them separated. This gives structural strength and stiffness to the bridge, making it a competent structure. The Helix is the first ever example of this structural solution applied to a significant bridge. It represents a design and engineering innovation, and is an entirely new classification of bridge structure.

Lateral synchronous vibration

One of the main concerns regarding dynamics during the design stage was that of the bridge's susceptibility to lateral synchronous vibration (LSV) – the phenomenon responsible for excessive vibrations felt on London's Millennium Bridge during its opening day[1]. When LSV occurs, pedestrians walking across a bridge progressively lock in step as they attempt to balance themselves laterally. This induces a build-up in vibration on the bridge deck and generates a lock-in phenomenon where the level of vibrations increases with time. Arup has contributed a substantial amount of research on the topic and recommends that the design fundamental frequency for the lateral mode be at least 1.5Hz.

The lateral stiffness of a pile was modelled together with the bridge structure in Strand 7 to take into account the soil–structure interaction effects. The stiffness of the pile would also affect the lateral stiffness of the bridge and hence was considered in the computation of the fundamental frequency for the lateral mode. The soil type surrounding the piles is predominantly marine clay – a very soft clay which has very little stiffness – which is common for structures around the Marina Bay area.

There are upper-bound and lower-bound solutions for the fundamental natural frequency of the lateral mode. The contribution of pile cap stiffness to the lateral stiffness of the support was one of the parameters examined. The design team adopted a lower-bound value for the bridge's design. The fundamental frequency for the lateral mode was computed to be 1.76Hz, which is higher than the recommended minimum of 1.5Hz. This meant that LSV would not occur on the Helix Bridge. Subsequently, on-site vibration measurement gave the fundamental frequency for the lateral mode to be 2.52Hz, which is much higher than the predicted frequency of 1.76Hz. The difference between predictions and measurement is most probably related to the theoretical presumption of low lateral stiffness of the pile and pile cap within the very soft clay.

Modal testing

Modal testing was carried out to determine the dynamic properties of the bridge. SysEng (Singapore) Pte Ltd was commissioned to undertake the modal testing. Professor James Brownjohn from Full Scale Dynamics Ltd was engaged by SysEng as a technical adviser for the modal testing.

The modal testing was conducted in the late evening over four days (24–27 November 2009) so that there was minimal impact on the ongoing construction work at the time of testing. There was a concern that construction activities would generate vibrations which would interfere with the results from the modal tests. The main contractor, Sato Kogyo, was therefore notified of the tests in advance and it was agreed that work on site would be prohibited during the testing period.

A test grid was developed for the bridge to enable efficient dynamic testing. The bridge was discretised into many numbered points (Figure 3), with each point representing a probable test location. The layout of the test grid was planned to efficiently capture symmetrical and asymmetrical vertical, lateral and torsional vibration properties for the three main spans. The test points located on the three pods would capture the vertical and torsional vibration properties.

The test aimed to ascertain the dynamic properties of the bridge and also the level of vibrations generated by small groups of people jumping – this was to simulate the behaviour of a jubilant crowd engaged in celebrative events on the bridge. Although this might be a design factor, it is not a normal one and is unlikely to result in any sustained synchronous response.

Test scenarios included: modal testing using frequency response function (FRF) measurements to controlled excitation by two mechanical shakers (APS 400) – these generate vibrations using statistically uncorrelated random signals; measurement of decay time–histories due to shaker shut-off; and measurement of groups of up to 20 people attempting to jump in unison to the beat of a metronome at selected points along the bridge deck. Ambient vibration was also measured on the bridge deck. These events were planned to take place at varying locations on the bridge spans and the cantilevered pod areas.

An array of 12 Honeywell QA750 servo accelerometers mounted on levelled Perspex base plates was used for the measurement of the vibrations on the bridge deck during the modal test. The shakers' input forces were measured using Endevco 7754A-1000 accelerometers attached to the inertial masses (Figure 4). All accelerometers are three-directional sensors and are able to capture accelerations in all three X, Y and Z directions.

In total, there were 43 test points in the grid, of which four were located at the abutments. Three pairs of reference test points were reserved on the bridge deck. Two test points were also mounted on each of the cantilevered pods. Since there were more test points than accelerometers, it was necessary to utilise roving response measurements – moving the accelerometers to new test locations between modal tests while holding at least one accelerometer to remain as a reference point for modal parameter computation.

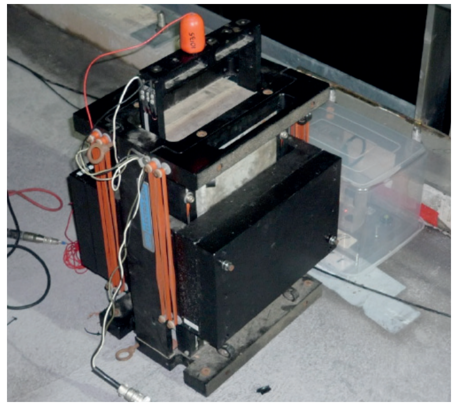
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Figure 4
APS 400 shaker and QA750 accelerometer (hidden in box)

Mechanical shakers excite a structure by forcing statistically uncorrelated random signals. They could excite the bridge structure to a high level of vibration which would give a good signal-to-noise ratio for the measurement data. Two sets of modal data could be obtained from each of the shaker shut-off tests: one set from the FRF measurements during the forced vibration phase and another set from the free vibration phase when the mechanical shaker was shut off and the forcing function became zero. A typical time–history plot is shown in Figure 5.

Analysis of modal test data

The measurement data from the modal tests were post-processed to give the corresponding eigenvalues and eigenvectors – natural frequencies and mode shapes of the bridge structure. The critical damping ratio was also calculated from the measurement data. These measured data were compared against earlier prediction values from the design stage (Table 1). The comparison of predicted and measured mode shapes is shown in Tables 2A and 2B.

Footfall analysis

Footfall analysis was performed on the Helix Bridge to ascertain that its dynamic performance against pedestrian footfall is satisfactory. This was carried out using the footfall analysis solver in Arup's in-house Oasys GSA software[2]. The solver assumes a single-person excitation approach for the computation in the footfall analysis. The solver also used input pedestrian parameters (single pedestrian with mass 76kg and walking on floor using Arup method – walking

TABLE 1: COMPARISON OF PREDICTED AND MEASURED MODAL PROPERTIES OF SIGNIFICANT MODES

Mode description	Predictions during design stage			As-built predictions			Measured		
	f (Hz)	Modal mass (t)	Damping (%crit)	*f* (Hz)	Modal mass (t)	Damping (%crit)	*f* (Hz)	Modal mass (t)	Damping (%crit)
First vertical, all spans (FV1)	1.69	306	0.8	1.91	282	0.8	1.90	277	0.5
Second vertical, all spans (FV2)	1.79	443	0.8	2.17	271	0.8	2.10	209	0.4
Third vertical, all spans (FV3)	2.10	104	0.8	2.42	230	0.8	2.30	252	0.7
First approach span, vertical (pod 4 side dominant) (FV5)	2.27	30.9	0.8	2.72	122†	0.8	2.82	53	1.5
Second approach span, vertical (pod 1 side dominant) (FV6)	2.36	37.5	0.8	2.74	97†	0.8	2.87	43	0.3
First torsional mode, all spans (FV7)	2.45	74.6	0.8	2.77	46	0.8	3.14	257	1.7
First main span, second bending mode (FV11)	3.30	217	0.8	3.61	115	0.8	3.63	236	1.0
First lateral – includes some lateral motion at piers (FL3)	1.76	301	0.8	1.87	530	0.8	2.52	899	1.3
Second lateral – includes some lateral motion at piers (FL4)	1.87	449	0.8	2.17	563	0.8	2.61	472	2.1
First pod only, vertical bending mode (FVP$_3$1)	2.8	*	0.8	3.0	*	0.8	3.3	45	2.2
First pod only, torsional mode (FTP$_3$1)	4.8	*	0.8	3.9	*	0.8	4.2	51	1.1

* Due to close matching of frequencies between pods, modal masses from model include multiple pods
† Both approach-span-only modes predicted contain significant participation from both approach spans, resulting in higher reported modal masses than in tests

TABLE 2A: COMPARISON OF PREDICTED AND MEASURED MODE SHAPES – VERTICAL

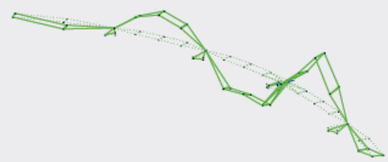
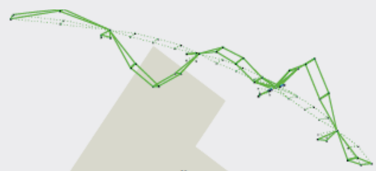
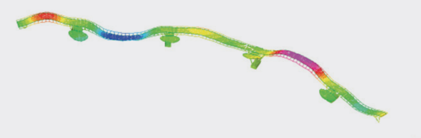
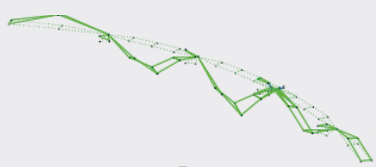
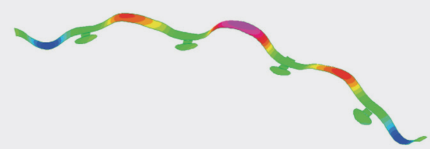
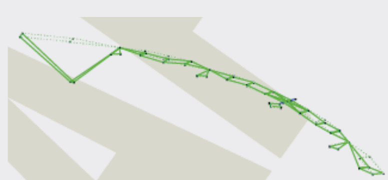
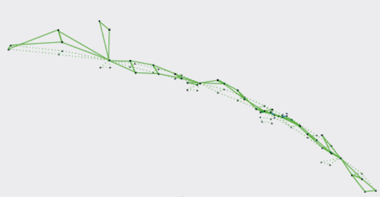
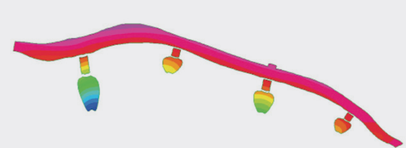
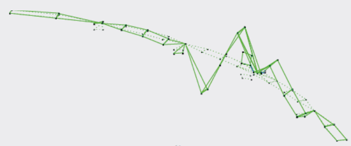
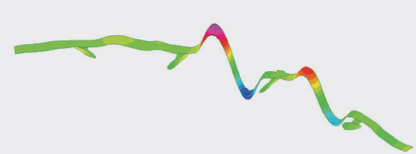
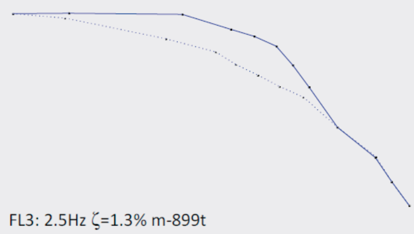
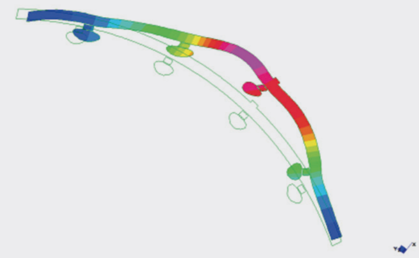
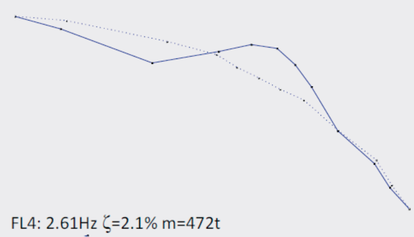
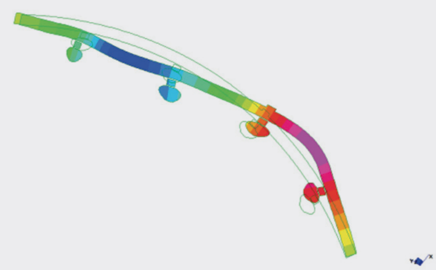
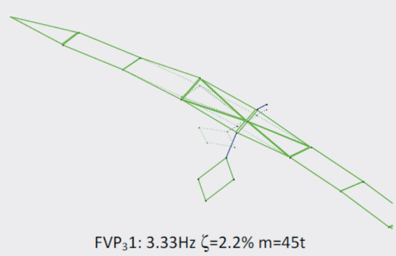
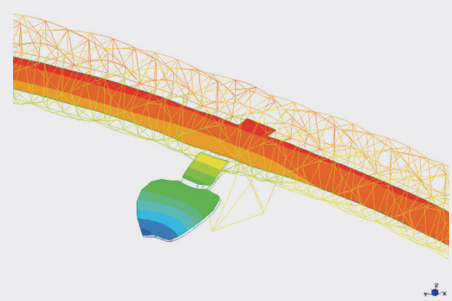
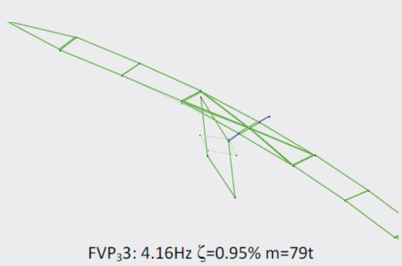
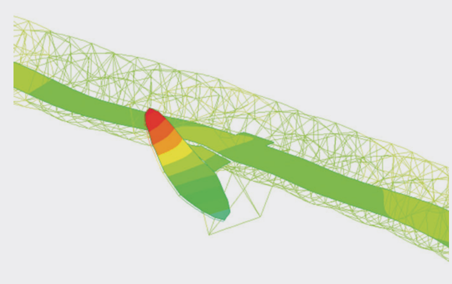
Mode	Measured mode shape	Predicted mode shape
1st vertical, all spans	 FV1: 1.91Hz ζ=1.0% m=277t	
2nd vertical, all spans	 FV2: 2.12Hz ζ=1.2% m=209t	
3rd vertical, all spans	 FV3: 2.31Hz ζ=1.0% m=252t	
1st vertical, approach spans only	 FV5: 2.82Hz ζ=1.5% m=53t	
2nd vertical, approach spans only	 FV6: 2.87Hz ζ=0.3% m=47t	
1st torsional, all spans	 FV7: 3.14Hz ζ=0.7% m=257t	
1st main span, second order bending mode	 FV11: 3.63Hz ζ=1.0% m=236	

TABLE 2B: COMPARISON OF PREDICTED AND MEASURED MODE SHAPES – LATERAL AND POD MODES

Mode	Measured mode shape	Predicted mode shape
1st lateral*		
2nd lateral*		
1st pod, vertical		
1st pod, torsional		

* There were two modes – FL1 and FL2, which measured 1.91Hz and 2.32Hz respectively – with lateral mode shapes that did not match the predicted shapes and which were hence omitted. The differences indicate that as-built foundation stiffnesses and restraint at abutments may be greater than assumed in the model. Measured modes FL1 and FL2 were not considered significant lateral modes.

frequency range of 1–2.5Hz) from CCIP-016[3].

The force function for single-person footfall excitation may be represented by

$$F(t) = G\left\{1.0 + \sum_{h=1}^{H} r_h \sin\left(\frac{h2\pi}{T}t\right)\right\} \qquad (1)$$

where:
$F(t)$ is the periodic footfall loads
G is the body weight of the individual in kilograms
h is the number of Fourier terms
r_h is the Fourier coefficient (or dynamic load factor, DLF)
T is the period of the footfall
H is the number of Fourier (harmonic terms) to be considered (4 is used for walking on floor using Arup method, 3 is used for walking on floor using SCI method, and 2 is used for walking on stairs).

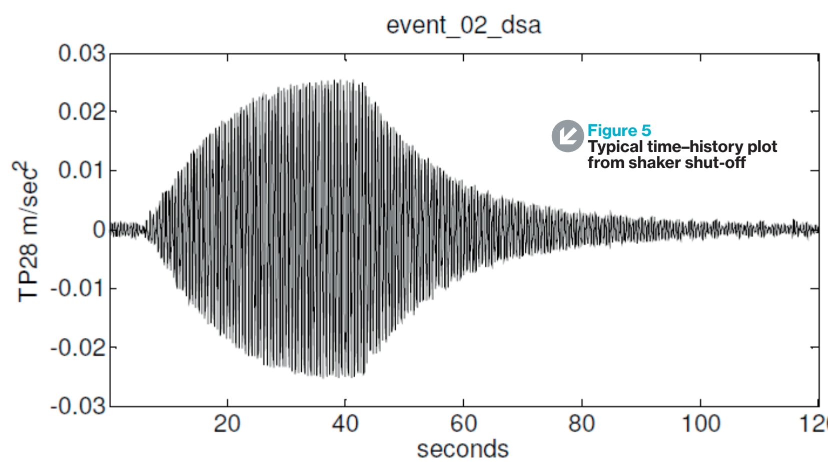


Figure 5
Typical time–history plot from shaker shut-off

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Vibrations for human comfort are generally measured using response factors (R-factors). Response factor is the ratio between the acceleration from the vibration generated and the threshold of barely perceptible vibration. For example, if the response factor is 2, the vibration is twice that of the barely perceptible vibration level. Weighted factors (from BS 6841[4]) are also generally applied to the accelerations measured as a correction factor for human comfort – human perception is more sensitive to particular ranges of frequencies and the weighted factor corrects for this phenomenon. Hence, weighted acceleration values are used for the comparison of human comfort for such vibration measurements.

Single-person excitation is the predominant method used to assess the dynamic performance of a footbridge. The method compares the predicted level of vibration caused by a single person walking at the most critical footfall rate. Many codes explicitly recognise this as the basis for their code checks. This method has generally ensured adequate vertical dynamic performance of external footbridges over the last 50 years. Calculations based on single-person excitation at the most critical footfall rate are satisfactory because the probability of more than one person walking at exactly the same speed, footfall rate and phase is extremely low and it is very difficult for a single person walking to produce the theoretical calculated level of response. Resonance of structures having low damping levels, such as footbridges, is extremely sensitive to footfall rate. The analysis output result is in the form of an R-factor.

Dynamic response

Numerical studies were carried out to determine the dynamic performance of the bridge. These studies were mostly referenced to vertical response with respect to single, walking pedestrians. A contour plot of the vertical vibration response factor at any point on the bridge deck due to a single person walking is shown in Figure 6.

The maximum response expected on the main deck of the footbridge is R = 29. This response is only predicted in the centre of the smaller approach spans. The response of the main spans of the footbridge is expected to be lower, no more than R = 10.

On the viewing decks, it is not possible for people to take more than 20 steps; therefore, only a proportion of the maximum steady-state response can be achieved. The response predicted on the balconies is shown in Figure 7. The maximum expected response at the side edge of the viewing deck is R = 32.

A comparison between measured and predicted responses is shown in Table 3. The measured vibration levels of the bridge are similar to those predicted, with the exception of the approach spans, which perform better than predictions. This is because predicted responses are based on a lower-bound estimation of stiffness of 2.27Hz, compared with a measured stiffness of 2.82Hz. The latter is higher than the possible walking step frequency, thus reducing the vibration levels on the approach spans significantly.

A study of vibration due to pedestrian crowd traffic was carried out on the bridge. Approx. 280 volunteers from the Land Transport Authority were involved in simulation tests of various activities that are likely to occur on the bridge. These include:

- a crowd walking in a closely packed fashion
- a crowd walking briskly
- a crowd jogging at a random pace
- separate crowds jogging along the inner edge of the bridge and walking along the outer edge
- separate crowds walking along the inner edge of the bridge and jogging along the outer edge.

Responses to the above scenarios were measured and evaluated for satisfaction of comfort criteria from the following international codes:

- Sétra guide[6] / JRC-ECCS[7] (used for a crowd jumping, walking and running)
- BS EN 1991-2 (Eurocode 1, Part 2)[8] (UK National Annex, used for a crowd walking)
- BS 5400[9] / DIN102[10] / AS 5100.2[11] (used for a single, walking pedestrian)
- ISO 10137[5] (used for a single, walking pedestrian)

The setting-out of the test points is indicated in Figure 8. Measured peak acceleration of all the test points (except test points 8 and 12 due to a dismounted accelerometer) under the simulated activities was plotted against the comfort criteria set out in BS 5400, DIN102, AS 5100.2, the Sétra guide and BS EN 1991-2 and is shown in Figure 9.

As can be seen from the vibration levels (Fig. 9) at locations in Fig. 8, the approach span (test points 2 and 3) exhibits the highest response during the running tests. Nevertheless, this is still well within the Sétra criteria for maximum comfort, which are more stringent than those in BS 5400, DIN102, AS 5100.2 and BS EN 1991-2.

ISO 10137 gives frequency-weighted acceleration criteria for a single, walking pedestrian on an outdoor and indoor footbridge. A plot of the criteria and test points is shown in Figure 10.

As can be seen, all weighted accelerations recorded for a crowd walking fall within the criteria stipulated for an indoor footbridge. Only weighted accelerations recorded for a combined crowd walking and running fall slightly above the criteria for an indoor footbridge and for a single, walking pedestrian.

Based on the comparison study for both walking and running, it can be seen that the recorded acceleration values are well within the comfort criteria and acceleration limits stipulated by the various international codes. It is worth noting that accelerations are well within the most stringent Sétra criterion even for tests which simulated the Singapore Marathon or Big Walk (mass-participation walk) events. Therefore, all acceleration

TABLE 3: COMPARISON OF PREDICTED AND MEASURED RESPONSES TO SINGLE PERSON WALKING

Location	Maximum predicted response factor	Maximum measured response factor	Acceptability criteria*
Main spans	8	7	64
Approach spans	29	6	64
Viewing platforms	20	22	32

* ISO 10137 footbridge criteria for walking pedestrians[5]

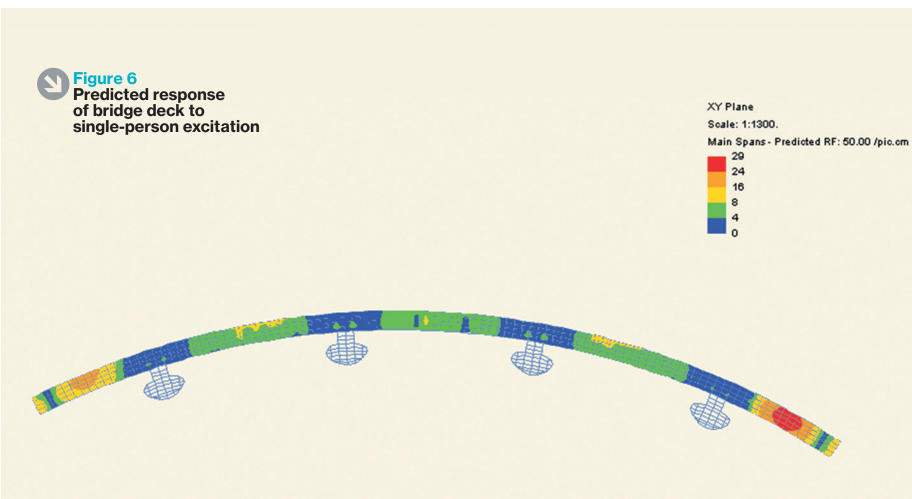

Figure 6 Predicted response of bridge deck to single-person excitation

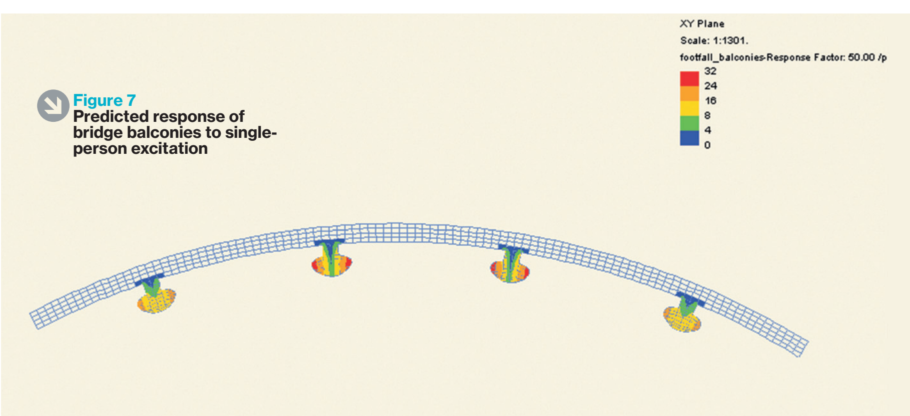

Figure 7 Predicted response of bridge balconies to single-person excitation

values recorded during the tests are considered acceptable.

Conclusions

The fundamental frequency for the first vertical mode was measured to be 1.9Hz, which was slightly above the predicted value of 1.69Hz. This could be due to the stiffness contribution of the architectural finishes and the facade items on the bridge. A comparison of the measured and predicted values for the natural frequencies for the majority of modes showed close agreement between the two, which suggested that the stiffness and mass were predicted to a good level of accuracy during the design stage.

Structural damping is generally difficult to predict for structures, as much damping results from friction and slip, and from connections and cladding. The modal data showed that the measured structural damping varies from 0.4% to 2.2% of critical damping depending on the mode.

The modal properties of a majority of the contributing modes of the Helix Bridge have been identified using a number of modal tests that included shaker-generated FRF measurements and human excitation. These methods gave a reasonable signal-to-noise ratio for identification purposes and the measured natural frequencies and mode shapes were very similar to those predicted during the design stage. The measured fundamental natural frequency for the lateral mode was found to be 2.52Hz, which is above the recommended design value of 1.5Hz; hence, the bridge would not be susceptible to LSV. In summary, the results from modal tests were in line with the original predicted values and thus confirmed the validity of the initial modal data used for the design of the complex footbridge structure.

The dynamic response of the Helix Bridge was measured under a scenario of different tests comprising various activities, e.g. walking and running. The comprehensive test programme also included the above-mentioned activities under different pedestrian group sizes. The measured results were found to be in close agreement with the predicted response of the bridge. It was also shown that the level of dynamic response measured was acceptable in accordance with the latest international codes.

Acknowledgements

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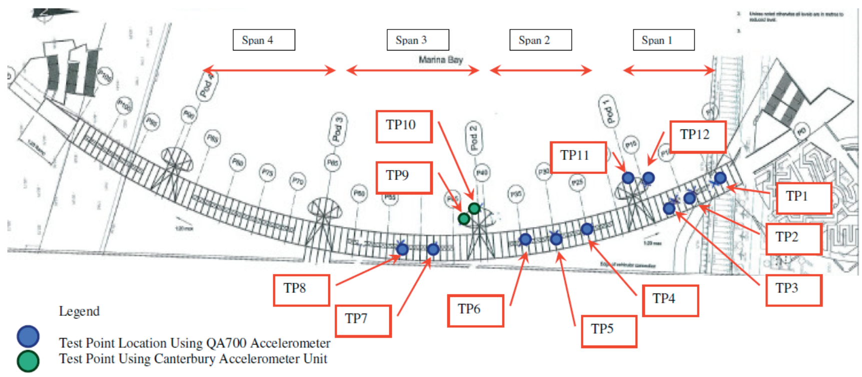
Figure 8
Setting-out of test points for measurement



Note: 1. Test Point TP1 to TP8 are placed on lower I Beam support of the bridge deck. Accessible only from service catwalk from maintenance manhole
2. Test Point TP9 and TP10 placed on Pod 2 deck. Test Point TP11 and T12 placed on Pod 1 deck.

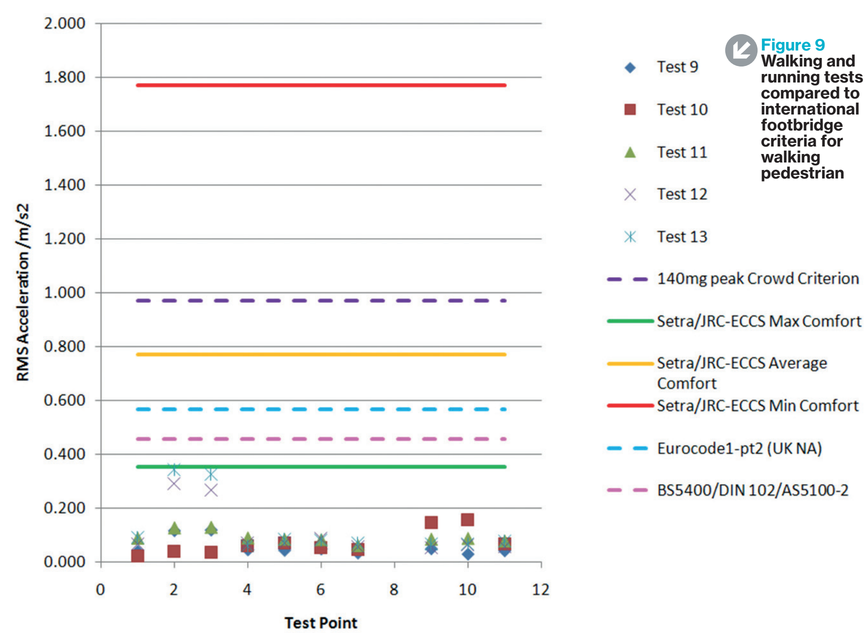


Figure 9
Walking and running tests compared to international footbridge criteria for walking pedestrian

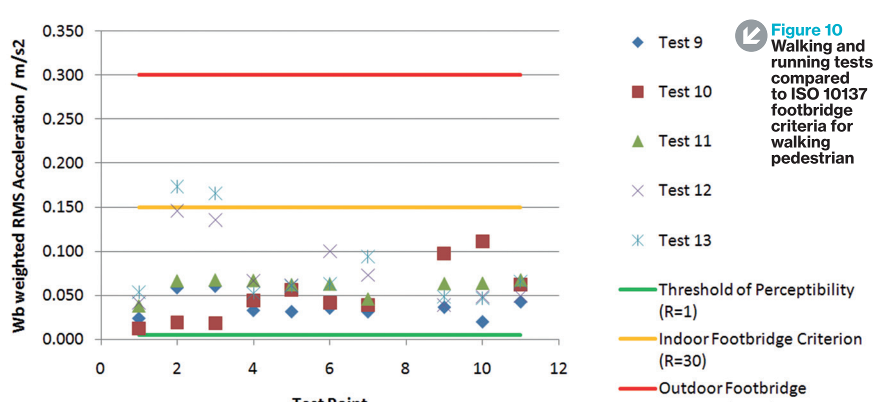


Figure 10
Walking and running tests compared to ISO 10137 footbridge criteria for walking pedestrian

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