Assessment of masonry bell tower response to bell ringing using operational modal analysis and numerical modelling

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ABSTRACT

The authors carried out an investigation into the dynamic response of a bell tower. The main focus of this study was to investigate the mechanism behind the perceived high vibration response during bell ringing as well as to provide an assessment of the severity of vibration response with respect to risk of damage to the structure. A combination of operational modal analysis techniques (including a non-contact measurement option) and finite element modelling was used to analyse the dynamic response of the structure. This paper discusses the measurement and modelling techniques implemented in order to assess the effect of bell ringing to the tower structure as well as retrofit and monitoring strategies proposed in order to manage and monitor the motion of the tower for improved safety.

INTRODUCTION

St. Paul's Cathedral, in Bendigo Victoria, was constructed in 1868 with the bell tower being completed in 1873. The bell tower, which stands 33 m tall, was fitted with eight bells. Bell ringing was halted in 1880 due to structural safety concerns, instead only being chimed from that point on. The bells were subsequently re-cast in England and recommissioned in 1964. At this time the bell frame was installed at a lower level and the tower was locally strengthened around this area.

The re-cast bells were lighter and were installed with shaped headstocks to reduce the out of balance pendulum forces, i.e. the centre of mass of the bells were moved closer to the centre of rotation. However, subsequent to these modifications the tower still has had a history of perceived high vibration levels during bell ringing. In 2009 the local council declared the building unsafe for bell ringing or chiming until such time that relevant assessment and remedial works (if deemed necessary) had been completed.

The authors were commissioned by the Anglican Diocese of Bendigo to carry out an investigation into the dynamic response of the bell tower. The key focus of this study was to investigate the primary cause of the perceived high vibration response as well as to provide an assessment of the vibration severity due to bell ringing with respect to the risk of damage to the structure.

The general procedure adopted to achieve these aims was:

- Define the fundamental natural modes of vibration of the bell tower using operational modal analysis methods.
- Measure the vibration response of the bell tower and bell frame during bell ringing events.
- Identify the bell ringing pendulum frequencies.
- Develop a finite element (FE) model of the bell tower including the effects of significant existing defects.
- Measure the global displacement of the tower during testing using the IDS IBIS-S non-contact system for measurement validation purposes.
- Update and validate the FE model to match the measured modal response.
- Use the validated FE model to assess the dynamic response during bell ringing. (i.e. the measured vibration response was applied to the model and the dynamic stress field plotted).
- Assess the measured vibration response due to bell ringing in relation to relevant Australian and international standards.
- Following completion of the measurement and assessment process, develop a mitigation strategy for the perceived excessive levels of vibration if warranted.

Throughout this paper, bell 'ringing' refers to a method which involves rotation of the bells, as opposed to 'chiming', which involves striking the bell with minimal rotation of the bell mass.

MEASUREMENT PROCEDURE

Instrumentation Installation

The instrumentation used for this study is listed below (Table 1) Accelerometers were fixed at an array of locations (Figure 1) inside the bell tower such that the fundamental sway modes of vibration could be characterised in the longitudinal (X-Z Plane) and transverse (Y-Z Plane) directions in accordance with the co-ordinate system adopted for the measurement program (Figure 2).

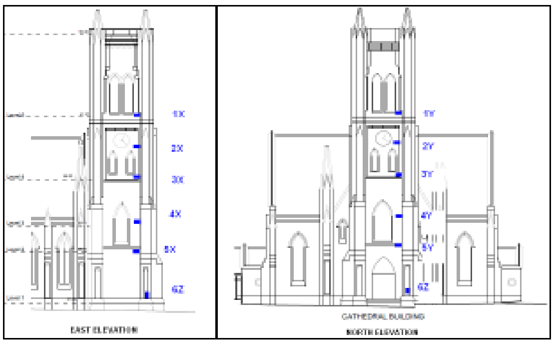

Figure 1. Accelerometer installation locations

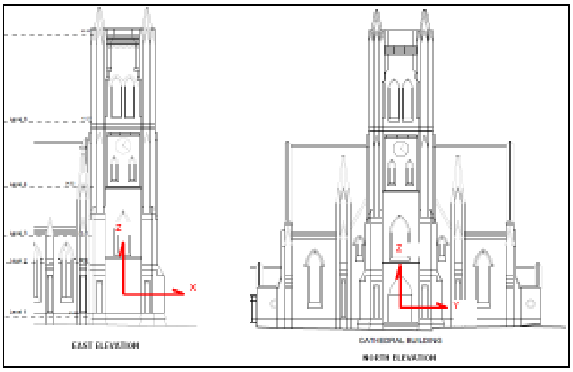

Figure 2. Elevations showing co-ordinate system

Table 1. List of Instrumentation

Instrument	Make	Type
Accelerometers	PCB	393A03
Data Acquisition	LMS	Scadas
Data Analysis / Processing	LMS	Test.Xpress Test.Lab
Finite Element Modelling	Ansys	Version 13
Non-Contact Displacement	IDS	IBIS-S

Natural Modes of the Bell Tower

In order to define the natural modes of the tower, measurements of tower vibration responses due to environmental excitations were recorded. In other words, background vibration response due to wind and/or ground-borne traffic induced vibration was measured.

These measurements were then used to extract the modal parameters of the first two natural modes of the tower in the X-Z and Y-Z plane, which are the fundamental and second order sway modes in the X and Y direction respectively.

Bell Ringing Induced Vibration

Operational vibratory response measurements (Table 2) were also carried out during a series of bell ringing trials. This was the first time the bells had been rung since 2009. The purpose of these measurements was to determine the level of vibration at various locations within the tower for different bell ringing configurations.

Table 2. Summary of operational vibration measurements

Test No.	Bells in Operation	Measurement Direction	Ringing or Chiming
1	No.8	X-Z	Ringing
2	No.7	X-Z	Ringing
3	No.6	X-Z	Ringing
4	No.5	X-Z	Ringing
5	No.4	X-Z	Ringing
6	No.3	X-Z	Ringing
7	No.2	X-Z	Ringing
8	No.2,3,4,5 ,6,7,8	X-Z	Ringing
9	No.8	Y-Z	Ringing
10	No.7	Y-Z	Ringing
11	No.6	Y-Z	Ringing
12	No.5	Y-Z	Ringing
13	No.4	Y-Z	Ringing
14	No.3	Y-Z	Ringing
15	No.2	Y-Z	Ringing
16	No.2,3,4,5 ,6,7,8	Y-Z	Chiming
17	No.2,3,4,5 ,6,7,8	Y-Z	Chiming

MODAL ANALYSIS RESULTS

Modal Parameters

The natural modes and modal parameters (Table 3) of the tower were defined on the basis of the ambient modal measurement results.

Analysis of the narrowband spectra for each measurement plane (Figure 3 and Figure 4) indicated the natural modes and allowed for modal parameters such as damping to be determined.

The fundamental natural sway modes of the tower were measured at approximately 1.4 Hz in the longitudinal direction and 1.6 Hz in the transverse direction.

Table 3. Summary of modal parameters

Mode No.	Frequency (Hz)	Damping (%)	Description of Mode Shape
1	1.43	3.3	Fundamental sway X-Z plane
2	1.56	3.4	Fundamental sway Y-Z plane
3	4.1	2.9	Second order sway X-Z plane
4	4.6	2.8	Second order sway Y-Z plane

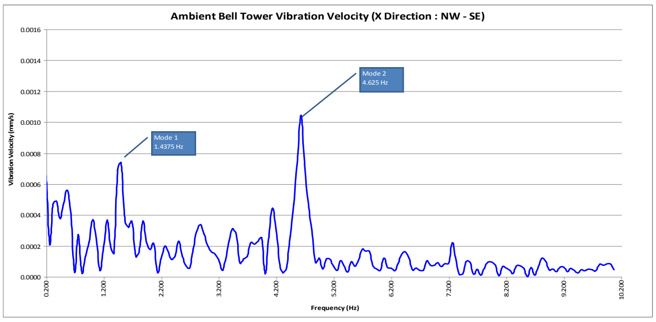

Figure 3. Sample ambient vibration spectrum (X-Z Plane)

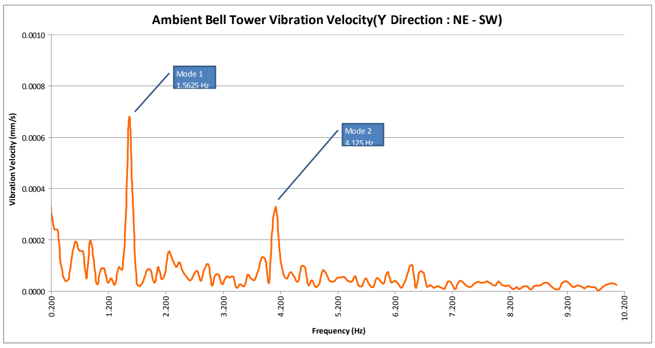

Figure 4. Sample ambient vibration spectrum (Y-Z Plane)

Mode Shapes

The natural mode shapes for each measurement were mapped onto a line drawing of the bell tower and the resulting mode shapes plotted and animated for viewing (Figure 5 and Figure 6) and for comparison with the FE model results.

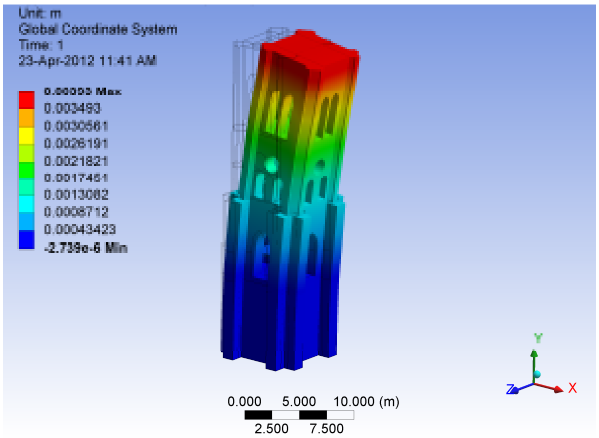

Figure 5. Mode 1 (1.4 Hz) – fundamental X-Z sway mode

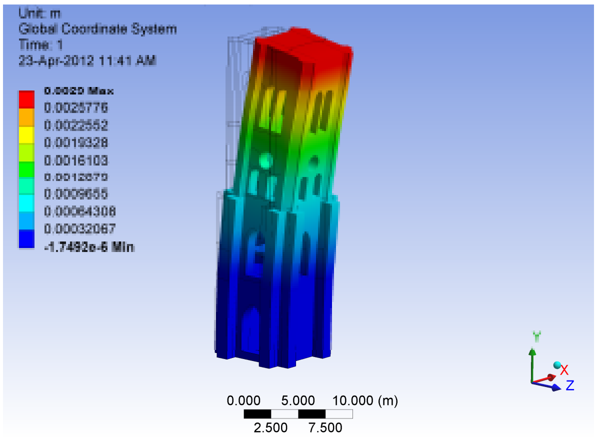

Figure 6. Mode 2 (1.5 Hz) – fundamental Y-Z sway mode

VIBRATION MEASUREMENT RESULTS

The displacement time histories (Figure 7 and Figure 8) for each accelerometer location whilst all bells were ringing showed that the displacement levels increased with increase in the height of the measurement location, an indication that the bell ringing was exciting the fundamental sway modes in each measurement plane.

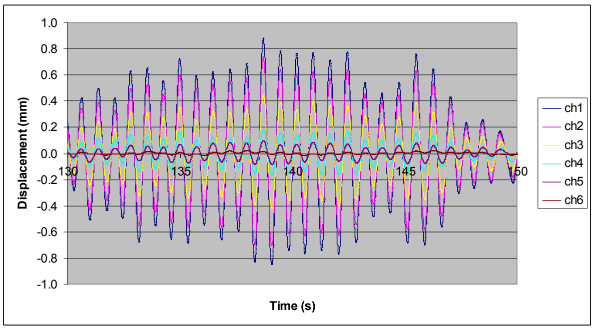

Figure 7. Sample displacement time history (All bells X-Z)

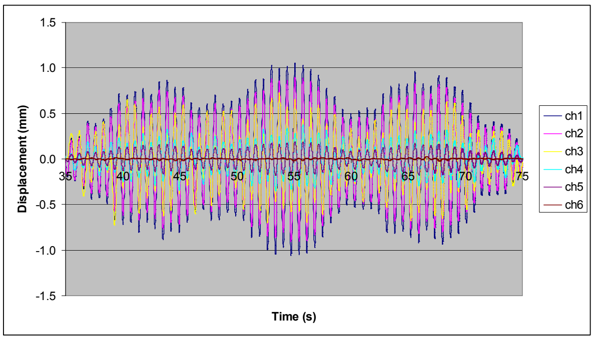

Figure 8. Mode 2 (1.5 Hz) – fundamental Y-Z sway mode

The vibration measurement results were analysed to provide the maximum vibration velocity and displacement for each test (Table 3).

Table 3. Summary of vibration levels and frequencies

Test No.	Maximum Vibration Velocity Level (mm/s RMS)	Maximum Vibration Displacement Level (mm Peak)	Dominant Frequency (Hz)
1	3.4	0.6	1.3
2	3.1	0.5	1.4
3	0.6	0.1	1.4
4	0.6	0.1	1.3
5	0.4	0.06	1.4
6	0.3	0.05	1.3
7	0.3	0.05	1.3
8	5.5	0.9	1.4
9	1.7	0.3	1.3
10	1.8	0.3	1.4
11	3.1	0.5	1.4
12	2.3	0.4	1.3
13	2.5	0.4	1.4
14	1.7	0.3	1.3
15	1.1	0.2	1.3
16	6.8	1.1	1.4
17	0.1	0.01	1.4

In all cases the maximum level was measured at the highest location in the tower.

It should be noted that the maximum measured vibration displacement was 1.1 mm when all bells were ringing (due to reasons of mechanical repair the smallest bell, Bell 1, could not be rung).

Due to poor access and risks to safety, it was not possible to mount accelerometers at the top level of the tower. Therefore the measured fundamental mode shapes were used to predict what the maximum displacement would be at the top of the tower with reference to the level measured at the highest accelerometer installation locations.

The maximum displacement level at the top of the tower due to vibration was extrapolated to be just below 2 mm (peak). This result correlates well with the highest displacement measured with the remote (IBIS-S) dynamic displacement measurement system (Figure 9).

The vibration measurements also found that there was no detectable relative motion between the steel bell frame and the masonry structure of the tower near the attachment points.

REMOTE DYNAMIC DISPLACEMENT MEASUREMENTS

The IBIS-S interferometric radar based displacement measurement system (Figure 9) was set up across the street from the bell tower on the top level of a two storey car park in order to avoid interference from other moving obstacles such as trees and vehicles. The system was directed at the bell tower (ensuring that the entire tower fit within the field of view of the system). IBIS-S displacement measurements (Figure 10 shows a sample displacement time history) were undertaken simultaneously to the accelerometer measurements (Table 2).

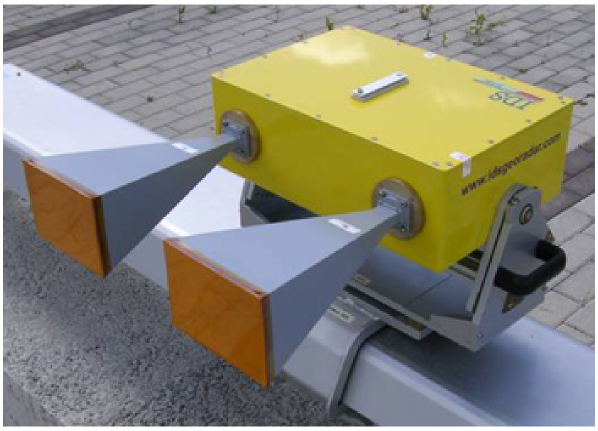

Figure 9. IBIS-S displacement measurement system

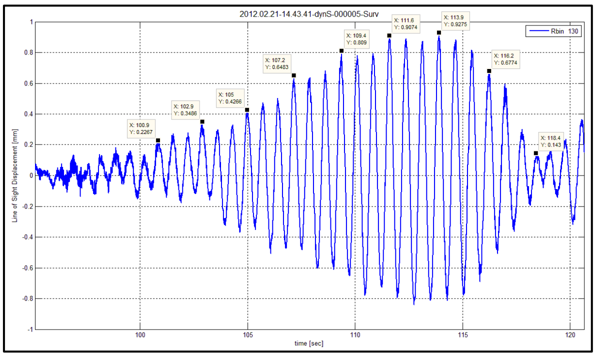

Figure 10. Sample IBIS-S displacement measurement

The results indicate that the maximum displacement at the top of the tower was 2 mm peak measured in the Y direction with all bells ringing.

FINITE ELEMENT MODEL

Model Description

A finite element (FE) model of the bell tower was prepared using ANSYS Workbench 13.0.

Higher order three dimensional brick elements were used to represent the masonry structure. The initial material properties (Table 4) used were based upon a literature review of similar masonry bell towers as an overall material property of the combined brickwork and mortar, treating it as a homogenous material (Ivorra et al., 2011).

The material properties were then updated (Table 5) to include the effects of cracks (determined via visual inspection and photogrammetry), directionality of the tower modal response (asymmetrical response measured) and the measured displacement amplitudes.

Table 4. FE Material Properties - Initial

Component	Density (kg/m^3)	Elastic Modulus (MPa)
Footing	2300	30000
Masonry brickwork (including mortar)	1800	2200
Bell Frame	2300	30000

Table 5. FE Material Properties - Updated

Component	Density (kg/m^3)	Elastic Modulus (MPa)
Footing	2300	30000
Masonry brickwork (including mortar) – X-Z direction	1800	2100
Masonry brickwork (including mortar) – Y-Z direction	1800	1000
Bell frame	IDS	IBIS-S

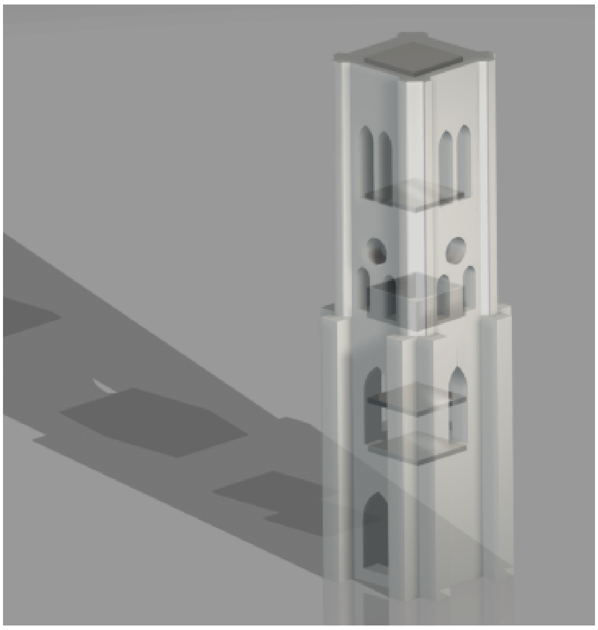

Figure 11. Geometry of the FE model

Existing Crack Modelling

The results of a photogrammetric survey (Figure 12) were used to simulate significant existing cracks in the FE model. The measured cracks were modelled physically (Figure 13) where they were found to be locally potentially significant, and in other areas they were modelled globally by incorporating them into the FE model by way of changed overall homogenous material properties.

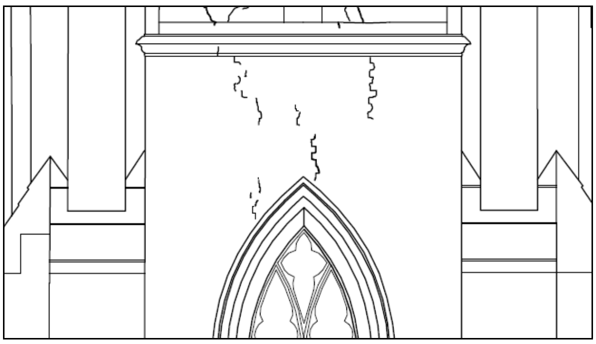

Figure 12. Sample plot showing documented cracks

Figure 13. View of FE model showing simulated cracks

Normal Mode Analysis and Model Refinement

Modal analyses were carried out using the FE model in order to validate the performance of the FE model against the measured vibration response that was obtained on site. The fundamental bending modes in both directions are shown below as resolved using the FE model including the updated material parameters (Table 5).

The resolved modes were 1.43 Hz and 1.54 Hz in the X-Z and Y-Z directions respectively (Figure 14 and Figure 15). These modes compare very closely to the measured vibration data, and provide a means of data validation.

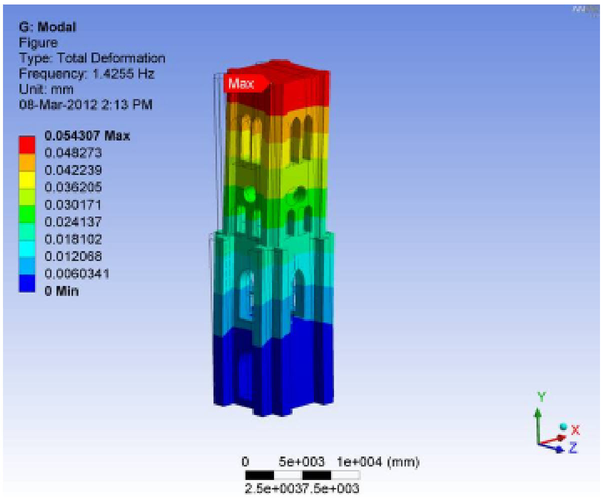

Figure 14. Fundamental bending mode (X-Z direction)

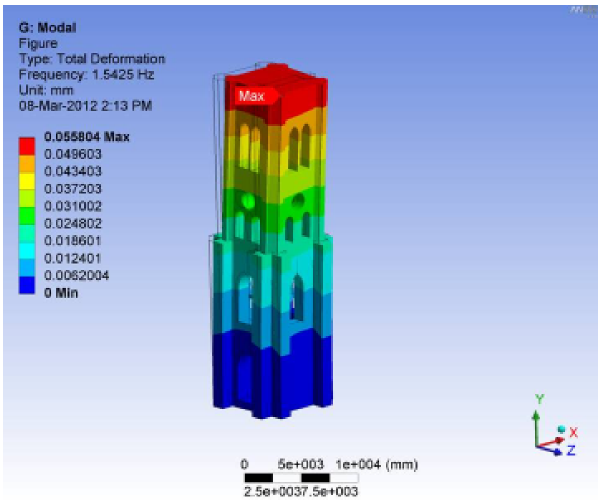

Figure 15. Fundamental bending mode (Y-Z direction)

Dead Load Analysis

A dead load analysis was carried out to find out the 'pre-compression' that the structure was likely to be under in its static state (Figure 15). It can be seen here that the highest stress is approximately 3.2 MPa in compression (found in the concrete collar).

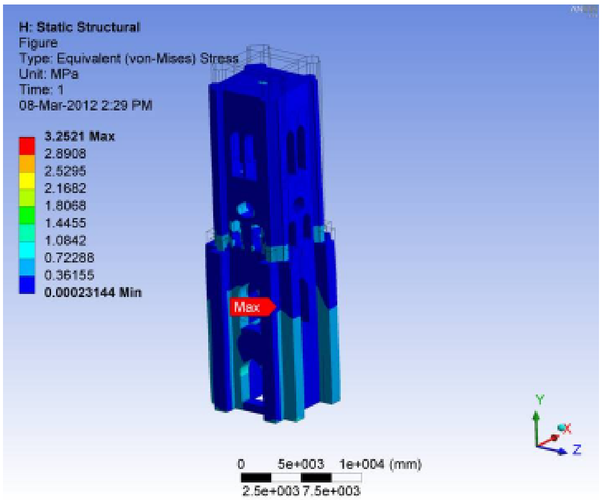

Figure 15. Stress contours from FE model dead load analysis

DISCUSSIONS

The author's literature review of bell towers and bell ringing dynamics led to the following observations:

- The nominal frequency range of the fundamental sway mode of bell towers is approximately 1.0 Hz to 2.4Hz.
- The nominal pendulum frequency of bell ringing mechanisms is in the range of 1.2 Hz to 1.6 Hz.
- The usual forced response behaviour of a tower under the action of one bell ringing is typically characterised by a short build up and slow decay as the bell swings through smaller and smaller arcs as evidenced by the sample displacement time history (Wilson and Selby, 1994) (Figure 16) which is in contrast to the build-up and decay of dynamic displacement observed at St Paul's (Figure 17).
- The difference in behaviour could be explained by the slow decay of the natural sway mode of the tower after it is excited by the almost sinusoidal input from the pendulum effect of the bell. The pendulum action of the bell, as it swings up, produces an increasing response in the tower, and once the bell has reached the bell-up position and ringing continues, the bell ringing occurs at a slightly lower frequency than it did at the start. Hence the forcing frequency beats with the natural decaying response of the tower.

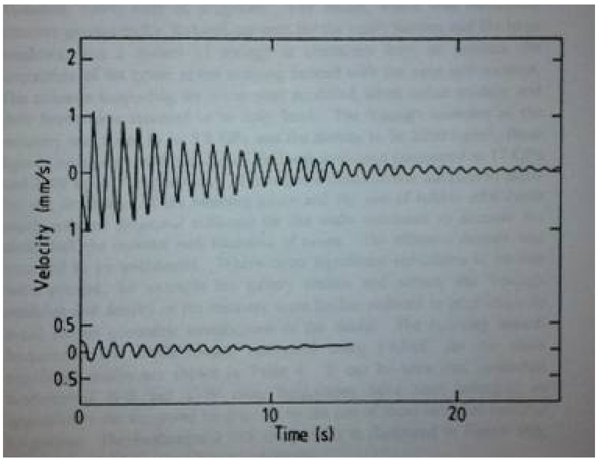

Figure 16. Sample displacement time history (Wilson and Selby, 1994)

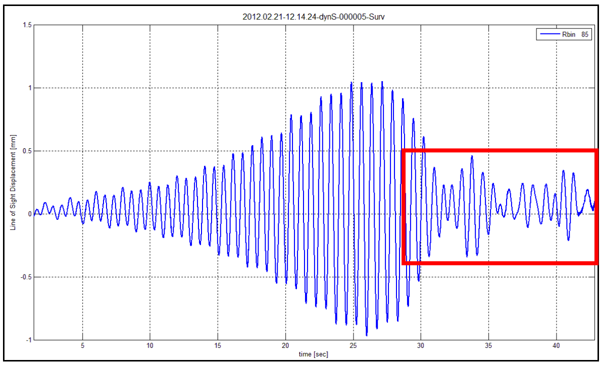

Figure 17. Sample displacement time history for Bell 8 at St Paul's Cathedral

VIBRATION SEVERITY STANDARDS

A survey of Australian and international vibration standards led the authors to conclude that the German standard DIN 4150-3 is the most relevant standard for this study. DIN 4150-3 provides "guidelines which, when complied with, will not result in damage that will have an adverse effect on the structure's serviceability". For this type of structure, the standard suggests a guideline criterion of 2.5 mm/s vibration velocity for continuous horizontal vibration. This assumes that the vibration due to bell ringing is classified as 'long-term' vibration as it has potential to cause "resonance in the structure being evaluated". DIN 4150-3 specifies this level to be compared against the measured vibration level in the horizontal direction on the top floor of the building.

It is very important to note however that this simple comparison does not ensure no damage will occur due to bell ringing. The standard notes that "exceeding the values… slightly does not necessarily lead to damage", and that "the stresses may be determined… shall be performed using state-of-the-art methods", which was carried out by the authors in this case.

DAMAGE RISK ASSESSMENT

Based on the above work and a survey of literature, the authros concluded that a damage risk assessment based on measured vibration levels alone, and compared with appropriate standards would not be considered accurate or effective in this case.

When the measured maximum dynamic displacements were applied to the validated FE model, the resulting maximum tensile stress developed in the masonry structure was 0.3 MPa (notwithstanding the existing dead load).

When the bells were chimed using the moving striker rather than rung, the vibration response of the tower was almost immeasurably low. The bells being chimed by the external striker method therefore poses no discernible risk to the structural integrity.

When the bells were rung with full rotations the maximum tensile stress determined at 0.3 MPa was less than the dead load stress at the same location; this means that the tower sway due to bell ringing would not cause overall tensile stress when the dead load is considered. In compression the tower sway will add a further 0.3 MPa to the existing compressive dead load of 0.5 MPa.

A review of existing published literature indicates that material strength of masonry varies between 2 MPa and 12 MPa in compression, and 0.3 and 0.5 MPa in tension (Ivorra et al, 2011). Results of this study indicate that the stresses imposed on St Paul's bell tower are less than the maximum allowable for masonry, according to the above properties.

This investigation had led us to the conclusion that while the reintroduction of bell ringing (both ringing and chiming) would introduce additional levels of stress to the tower, the stresses are likely to be below nominated material strengths of masonry typically used in bell tower structures. However, additional monitoring and mitigation measures are proposed to further reduce the risk of structural damage due to bell ringing. These measures may include:

- Modification of the bells to reduce the out of balance pendulum forces during ringing.
- Implementation of a tuned mass damper system within the tower to counteract the bell ringing induced vibration.
- Implementation of a monitoring system that would measure the motion of the tower (potentially including strain) on a continual basis to warn the tower operators of exceedances specificed safe limits.

These recommendations are made to offset the following points of concern.

Our research has indicated that historically, there have been sudden collapses of masonry towers (Gentile, 2007).

The response and stress calculations in our assessment were based on the maximum measured response during the trial ringing. It is now understood that the tower sway mode is close to the input pendulum frequencies, and it is speculated that slight changes in the pendulum frequencies may produce a higher (or lower) tower response. Namely, if the pendulum frequencies, measured to be slightly lower than the tower sway modes, were to coincide with the tower frequencies then a higher tower response may result.

Furthermore, variation to phasing of the bells could also potentially produce a higher or lower response. The trials undertaken were an attempt to measure the tower response due to typical ringing activites with no particular regard for phasing of bells.

CONCLUSIONS AND RECOMMENDATIONS

This paper presents the results of the above investigation of the dynamic response of the bell tower at the St Paul's Cathedral in Bendigo, Victoria.

The main conclusions of this investigation were:

- The perceived high vibration response of the bell tower during bell ringing was quantified.
- The highest measured vibration level was 6.8 mm/s RMS, measured at the upper level of the bell frame. This was measured whilst bells 2 to 8 were ringing.
- The fundamental natural sway modes of the tower were measured at 1.4 Hz in the longitudinal direction and 1.6 Hz in the transverse direction.
- It was found that the perceived high vibration response of the bell tower was due to resonance of the bell pendulum frequencies with the natural sway frequencies of the tower. The coincidence of the bell pendulum frequencies with the natural sway frequencies of the tower causes amplification of the vibration response in comparison to if there had not been a coincidence of frequencies.
- When the measured maximum dynamic displacements were applied to the validated FE model the resulting maximum tensile stress developed in the masonry structure was 0.3 MPa (notwithstanding the existing dead load).
- When the bells were chimed using the moving striker rather than rung, there was an almost immeasurably low vibration response of the tower. The bells being chimed by the external striker method therefore poses no discernible risk to the structural integrity.
- When the bells were rung with full rotations the maximum tensile stress (determined to be 0.3 MPa) was less than the dead load stress at the same location; this means that the tower sway due to bell ringing would not cause overall tensile stress when the dead load is considered. In compression the tower sway will add a further 0.3 MPa to the existing compressive dead load of 0.5 MPa.
- A review of existing published literature indicates that material strength of masonry varies between 2 MPa and 12 MPa in compression, and 0.3 and 0.5 MPa in tension (Ivorra et al, 2011). Results of this study indicate that the stresses imposed on St Paul's bell tower are less than the maximum allowable for masonry, according to the above properties.
- A potential next stage to this investigation is to further develop an appropriate monitoring and mitigation plan as described above.

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