Dynamic characterization of the DUOC-UC footbridge

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ABSTRACT: The results of an operational modal analysis conducted on a footbridge in Concepción, Chile, are presented in this article. Accelerometers were attached at different points on the footbridge and one-hour ambient tests were performed on different days and hour times. Stochastic Subspace Identification (SSI) and Frequency Domain Decomposition (FDD) were employed to determine the dynamic properties of the structure (modal frequencies and mode shapes). The experimental results were compared with the modal response calculated from a numerical model in SAP2000. Results are discussed in terms of the similitude between experimental and theoretical response.

1 INTRODUCTION

Footbridges are generally slender structures that may experience large displacements or excessive vibrations due to ambient excitations (pedestrian flow, vehicles traffic, wind, tremors, etc). These structures are usually designed using computational models that verify the strength of each of the constituent elements, but also the design must meet serviceability conditions, such as, limits for deformations and vibrations. Usually, strength conditions are easily satisfied by footbridge design. However, it is not exceptional finding structures that experienced vibrations which amplitude or frequency generates discomfort in users. This undesired structural behavior is usually not predicted by the numerical models because they do not necessarily represent the actual response of the structure in an exact manner (Bayraktar *et al*., 2010). Moreover, numerical models are generally not validated by any mean. Hence, it is impossible that these models can trustfully predict the actual structural performance if they are not calibrated using experimental measurements on the real structure.

A tool to calibrate numerical models considers measuring the low intensity vibrations that experienced structures due to ambient excitations (Ren *et al.*, 2004; Gentile *et al.*, 2011). The modal parameters that dynamically characterize the structures (frequencies, mode shapes and damping) are extracted from these operational vibration records. This information is then compared to the response predicted by numerical models and the models are adjusted to match the response measured on the real structure. These procedures are known as operational modal analysis (Brincker *et al*., 2000), system identification and model updating.

Our study aims to identify the modal properties (modal frequencies and mode shapes) of a pedestrian arch bridge built in laminated timber and steel. These experimental data will be then employed to obtain an improved numerical model of this structure that will be used to predict the bridge performance under operational and extreme conditions.

2 DESCRIPTION OF THE STRUCTURE

DUOC–UC footbridge is located over Avenida Paicaví, in front of the UCSC Medical Center in Concepción, Chile (Fig. 1). The footbridge consists of two 52 m long and 6.5 m height arches. The main elements of the arch are four curved beams made of laminated timber of radiata pine hinged at both ends (supports and middle length of the arches). These beams have a rectangular section of 1100 x 200 mm. A steel structure is attached to the main beams, with columns spaced every 4 m. This steel structure supports a 100 mm thick radiata pine slab with an 80 mm thick top layer of asphalt (Fig. 2).

This footbridge is highly transited because it connects two educational institutions (UCSC and DUOC-UC) with a main road. The peak of pedestrian flow occurs between 12:00 and 14:00 hours and between 17:00 and 19:00 during week days, especially from March to December. The peak of vehicular flow under the bridge occurs at the same time frame.

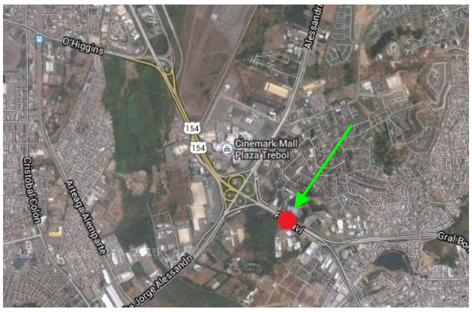


Figure 1: Reference location of footbridge in Concepción

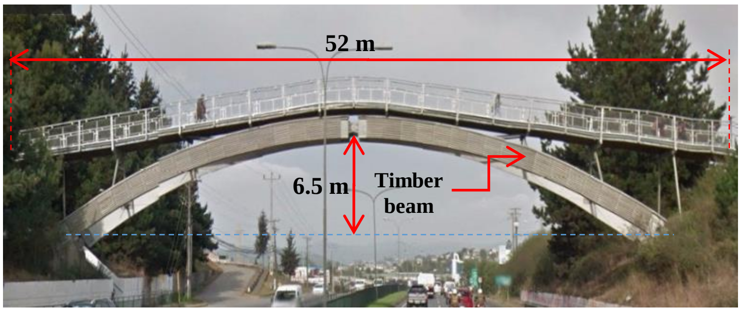


Figure 2: DUOC-UC footbridge

3 NUMERICAL MODEL

3.1 Model description

A model was generated using the finite elements software SAP2000 (Fig. 3). The numerical model has 256 nodes and consists of 330 frame elements and 102 shell type elements. The traffic slab, constituted of timber slab covered by an asphaltic layer, was modelled with shell element (100 mm thick) which equivalent to weight was 1520 kg/m^3. Timber mechanical properties (Young’s modulus and Poisson’s ratio) were assigned to the traffic slab as a first modelling approach. The properties of steel members, laminated timber beams and traffic slab are presented in Table 1. This model was used to determine the optimal locations for structure instrumentation.

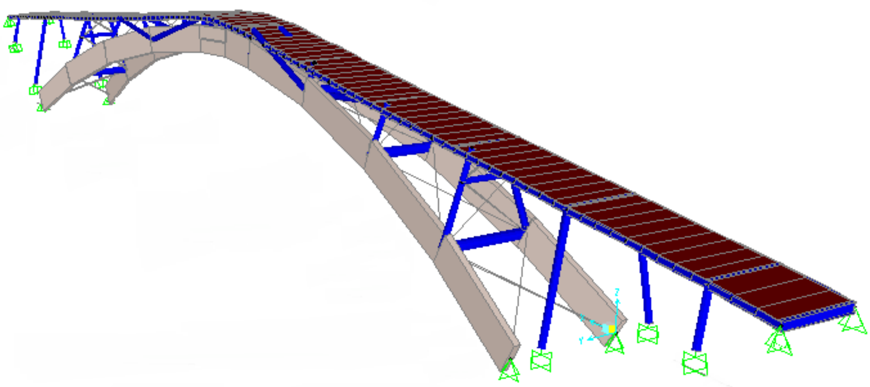


Figure 3: Footbridge model in SAP2000

Table 1: Material properties

	Laminated Timber	Steel	Traffic Slab
Young's Modulus (MPa)	112	2,1	104
Poisson's Ratio	0.3	0.3	0.3
Weight (kg/m^3)	480	7800	1520

3.2 Model results

The modes with higher participation of the traffic slab in the vertical direction were identified and the corresponding modal frequencies and mode shapes are presented in Figure 4. The modal displacements recorded at the optimal locations (A1 to A6) identified to determine the mode shapes are presented in Table 2. These locations were considered latter in this study to attach the instruments for experimentally identifying modal response.

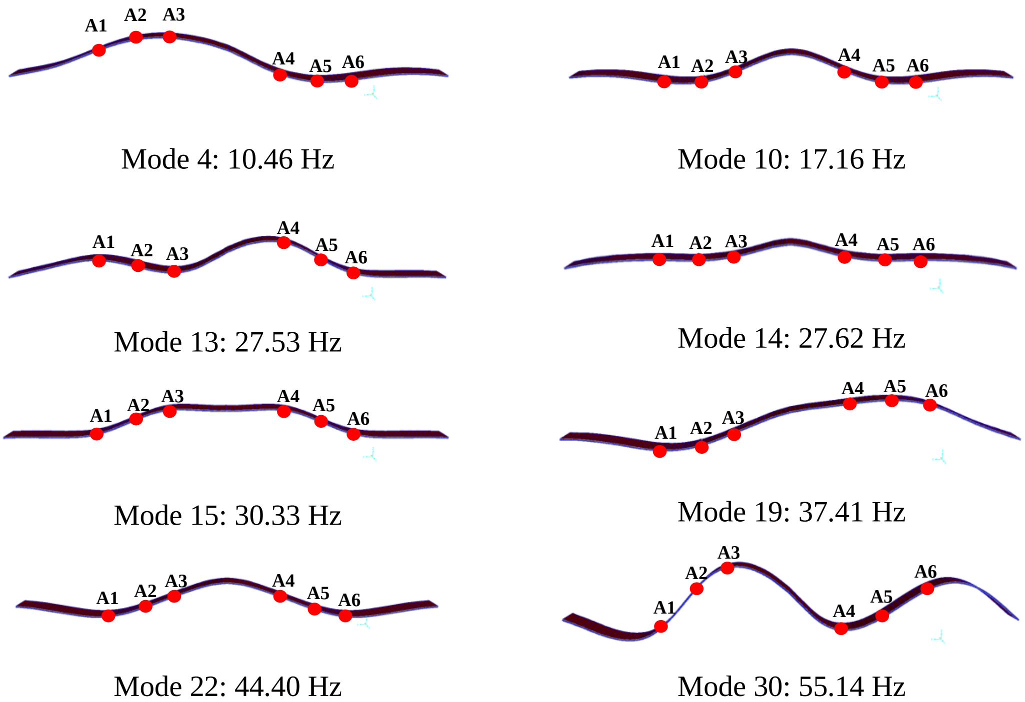


Figure 4: Results of the numerical mod

Table 2: Modal displacements at the identified degrees-of-freedom

	ϕ_{4}	ϕ_{10}	ϕ_{13}	ϕ_{14}	ϕ_{15}	ϕ_{19}	ϕ_{22}	ϕ_{30}
A1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
A2	1.42	1.24	-0.60	2.55	-0.15	1.01	0.80	-0.60
A3	1.25	0.83	-2.25	2.92	-1.00	0.63	0.41	-1.62
A4	-1.25	0.83	2.25	2.92	-1.00	-0.63	0.41	1.62
A5	-1.42	1.24	0.60	2.55	-0.15	-1.01	0.80	0.60
A6	-1.00	1.00	-1.00	1.00	1.00	-1.00	1.00	-1.00

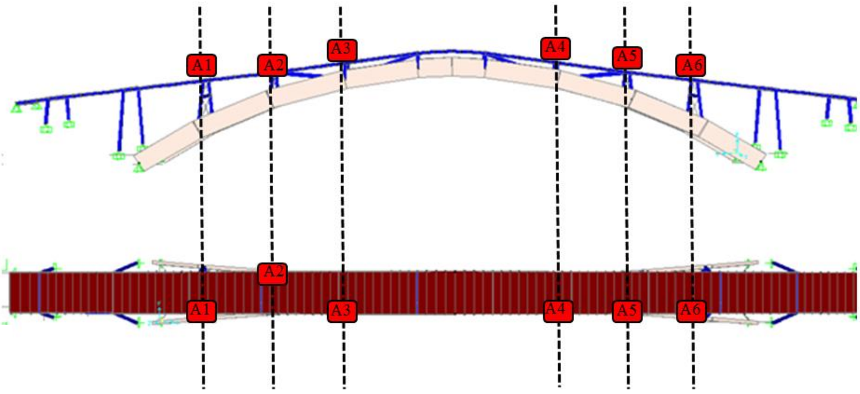
4 EXPERIMENTAL CAMPAIGN

4.1 Instrumentation

Measurements were recorded by accelerometers (Memsic CXL04GP3). These transducers are able to measure in the three main directions, with a sensitivity of 500 ± mV/g, in a range of +/- 4g up to 100 Hz. The data acquisition system is constituted by a NI9205 voltage module mounted on a cDAQ–9174 chassis, manufactured by National Instruments. The accelerometers were connected to the data acquisition system by multipolar cables which length range from 10 to 50 m. These cables were shielded with aluminium paper, in order to avoid a signal contamination from high a voltage network located in the vicinity of the footbridge. Data recording was controlled by a routine programmed on a LabView platform. Accelerometers were attached to the asphalt top-layer of the footbridge (Figure 5), avoiding to locate them on potential antinodes of the slab. This condition was verified using the numerical model. The measuring campaign was conducted in three different days as it is presented in Table 3.

Table 3: Information of data collection

	Test 1	**Test 2**	**Test 3**
Date	19/03/2015	06/04/2015	07/04/2015
Starting time	16:30	16:30	15:30
Finishing time	17:30	17:30	16:30



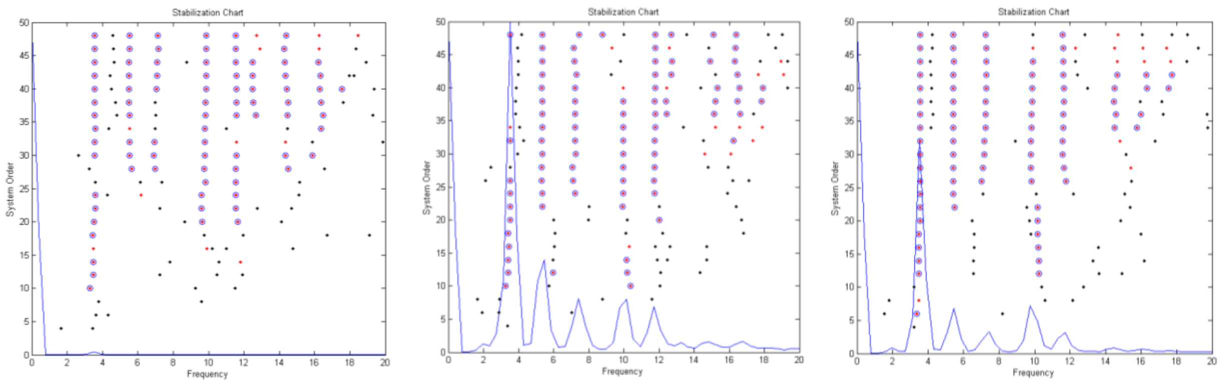
(a)

(b)

Figure 5: Position of the accelerometers. (a) Lateral view, and (b) Plane view.

4.2 Modal parameters identification

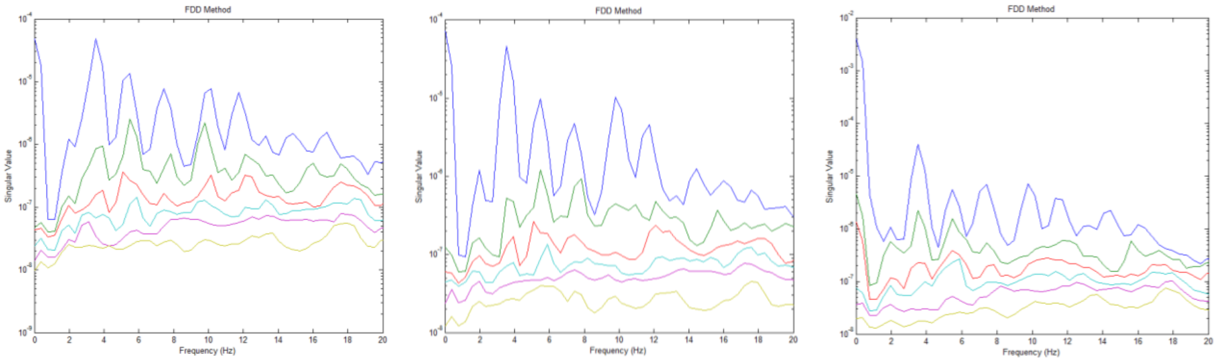
Two system identification methods were employed to determine the modal properties of the structure: Stochastic Subspace Identification (SSI) and Frequency Domain Decomposition (FDD). The SSI method (van Overschee & de Moor, 1996) is a data-driven time-domain technique that employs QR-factorization and singular value decomposition to identify the matrices of the dynamic state-space model. Once the state space model of the structure is found, the modal parameters (natural frequencies, damping ratios and mode shapes) can be determined by eigenvalues decomposition. In general, it is not possible to determine the system order beforehand. Therefore, it is necessary to repeat the analysis with different system orders and verify the repeatability of the results. This procedure is performed by constructing stabilization charts (Figure 6). In this graph, the dots represent the fundamental frequencies of the poles (modes) identified considering models with different system orders (*SO*). The red dots are associated with those frequencies that are similar to another frequency detected in the precedent model, while the blue circles around the dots represents those poles that have a similar mode shape to a pole detected in the precedent model. Those poles that reveal stability in terms of similar frequencies and mode shapes (usually aligned in a vertical column in the graph) are very likely to represent vibration modes.



(a) (b) (c)

Figure 6: Stabilization charts of (a) Test 1, (b) Test 2 and (c) Test 3.

The FDD method (Brincker *et al.*, 2000) is an extension of the classical peak-picking method. The FDD algorithm assumes that the excitation applied on the structure has a random nature and can be described as a white-noise. Thus, the excitation power spectral density function (PSD) becomes a constant (*S*) and, consequently, the FRF peaks can be directly identified from the peaks of the response PSD function. These peaks on the PSD function are assumed as resonant frequencies and mode shapes can be determined by applying Single Value Decomposition procedures. The PSD curves obtained for this experiment are presented in Figure 7.



(a) (b) (c)

Figure 7: Power spectral density curves of (a) Test 1, (b) Test 2 and (c) Test 3

The SSI and FDD methods were able to identify 9 modes. In general, the modal frequencies obtained by both methods coincide as can be observed in Table 4.

Modal assurance criterion (MAC) is used to compare experimental modal shapes obtained in the different test and those determined by different methods. MAC is an indicator that represents the degree of similitude between two mode shapes by determining the minimum square deviation (Eq. 1). Values close to unit indicate a high similitude between those two modes, and values close to zero do indicate no similitude between modes.

$$MAC(\phi_i, \phi_i^*) = \frac{|\phi_i^T \phi_i^*|^2}{(\phi_i^T \phi_i)(\phi_i^{*T} \phi_i^*)} \quad (1)$$

where ϕ_i= modal vector identified for testing i ; and ϕ_i^*= modal vector identified for testing i^*. The results of this comparison are presented in Tables 5. It can be observed that the modes identified in Test 1 are similar to those identified in Test 2, while the results of the comparison Test 1 vs Test 3 and Test 2 vs Test 3 are deficient.

The mode shapes obtained from the data extracted from Test 2 by SSI and FDD methods are compared in Figure 8.

Table 4: Experimental modal frequencies

Mode	SSI					FDD					Difference (%)
	Test 1 (Hz)	Test 2 (Hz)	Test 3 (Hz)	Average (Hz)	CoV (%)	Test 1 (Hz)	Test 2 (Hz)	Test 3 (Hz)	Average (Hz)	CoV (%)	
1	3.56	3.52	3.54	3.54	0.61	3.52	3.52	3.52	3.52	0.00	0.6
2	5.51	5.34	5.43	5.43	1.59	5.47	5.47	5.47	5.47	0.00	0.7
3	7.09	7.23	7.27	7.19	1.34	7.42	7.42	7.42	7.42	0.00	3.1
4	9.84	9.97	9.85	9.89	0.75	9.77	10.16	9.77	9.90	2.27	0.1
5	11.57	11.77	11.60	11.64	0.92	11.33	11.72	11.72	11.59	1.94	0.4
6	12.50	12.68	-	12.59	0.99	13.28	13.28	-	13.28	0.00	5.3
7	14.46	15.25	14.50	14.73	3.02	14.45	14.84	14.45	14.58	1.54	1.0
8	16.30	16.57	16.31	16.39	0.91	16.02	16.80	16.02	16.28	2.77	0.7
9	-	17.94	17.75	17.85	0.75	-	18.36	17.58	17.97	3.07	0.7

Table 5: MAC values

	SSI			FDD		
Mode	Test 1 vs Test 2	Test 1 vs Test 3	Test 2 vs Test 3	Test 1 vs Test 2	Test 1 vs Test 3	Test 2 vs Test 3
1	0.811	0.807	0.955	0.817	0.805	0.967
2	0.794	0.279	0.439	0.892	0.357	0.642
3	0.966	0.613	0.599	0.878	0.795	0.897
4	0.220	0.016	0.471	0.390	0.083	0.406
5	0.620	0.290	0.508	0.611	0.192	0.669
6	0.862	--	--	0.906	--	--
7	0.002	0.740	0.044	0.000	0.016	0.561
8	0.226	0.044	0.171	0.340	0.05	0.024
9	--	--	0.269	--	--	0.385

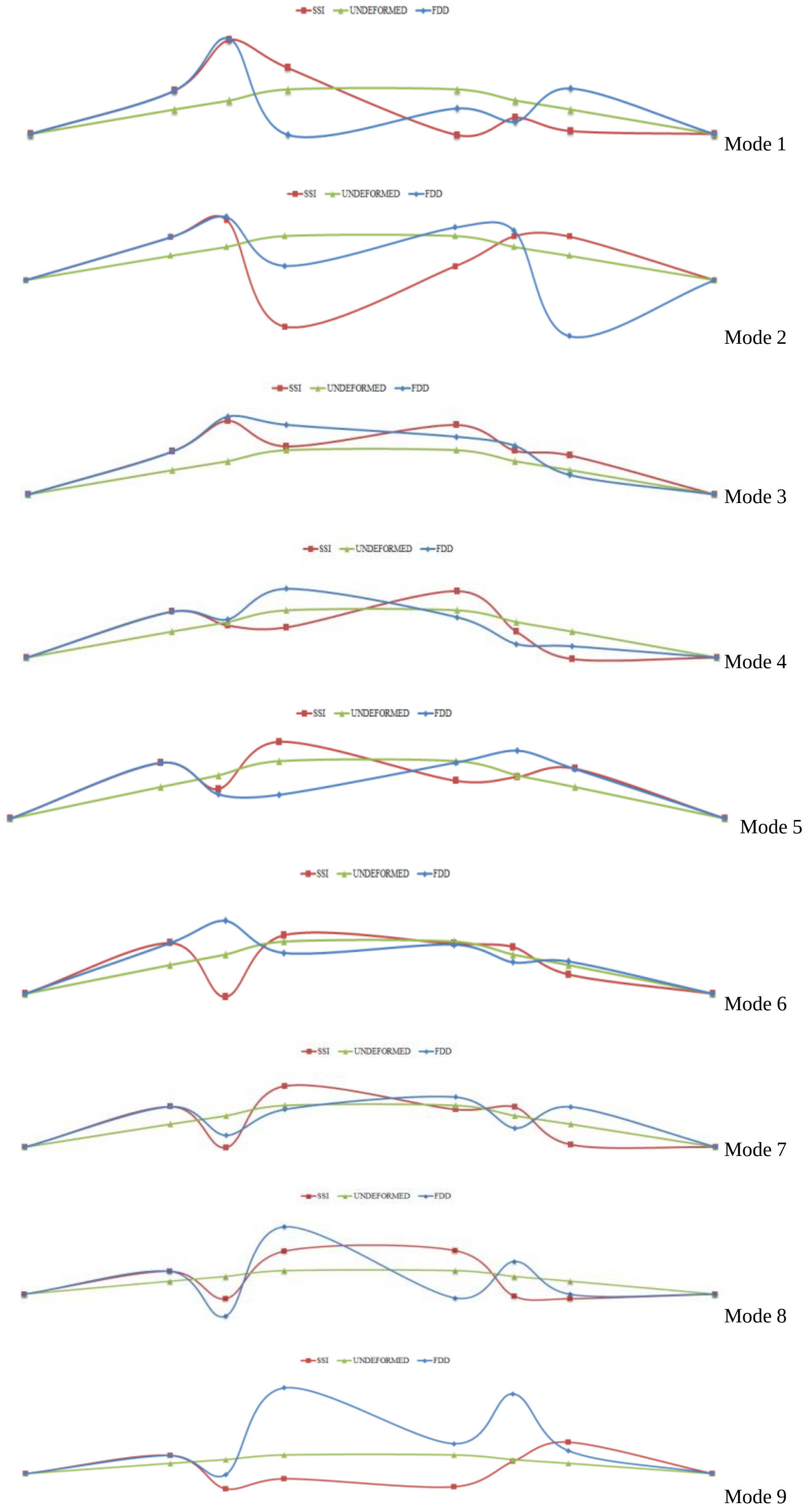
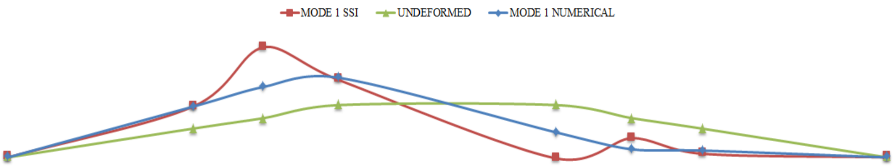


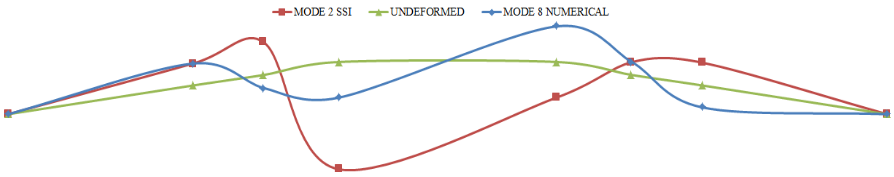
Figure 8: Experimental modal shapes

Tables 4 and 5 show that the first three detected modes are the most similar in terms of frequencies and shapes. The results obtained for these modes were visually paired with the modes numerically predicted by our SAP2000 model. This comparison is presented in Figure 9 and Table 6. The differences in between experimental and numerical frequencies are evident, even though the model was constructed using the most appropriate information available.

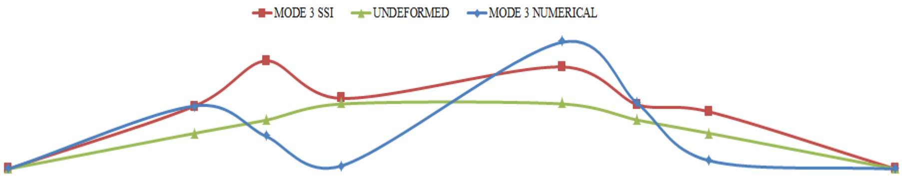
This situation clearly illustrate the driving idea presented in this paper: Numerical models must be calibrated using experimental measurements if an accurate representation of the actual structures behaviour is desired.



(a) Experimental mode 1 vs. Numerical mode 4



(b) Experimental mode 2 vs. Numerical mode 30



(c) Experimental mode 3 vs. Numerical mode 13

Figure 9: Comparison of experimental and numerical mode shapes

Table 6: Comparison of experimental and numerical frequencies

Experimental freq. (Hz)	**Numerical freq. (Hz)**
f_1=3.53	f_4=10.46
f_2=5.45	f_{30}=55.14
f_3=7.31	f_{13}=27.53

The experimentally identified frequencies and those numerically computed are presented in Table 6. Significant differences are observed between these two results that represent the same vibrating modes. These differences can be related to misestimations of the actual material properties considered for the numeric model. For example, one of these properties is the timber Young´s modulus that was obtained from Chilean Standard (NCh 1198 Of.2006). However, this property is sensitive to environmental conditions, such as, humidity and temperature, and it can be also affected by damage due to precedent earthquakes or other kind of extreme load. This estimation can be improved by performing experimental test on that material, but at the moment it was not possible to extract samples from the structure.

Even though, this modelling approach has followed the most common engineering practice to be applied for this kind of projects and was assisted by field surveys, it was not able to perfectly replicate the actual structural behaviour. This is something we should also expect in the process of design of any similar structure. Thus, it was demonstrated that experimental tests are essential to be performed for calibrating numerical models that provide accurate representations of the structural response.

5 CONCLUSIONS

Operational modal analysis was performed on an arched footbridge in Concepcion, Chile. Ambient vibrations due to pedestrian and vehicular traffic were considered as source of excitations. SSI and FDD methods were successfully employed to extract the modal properties of the structure.

SSI and FDD methods were able of identifying nine modal frequencies from Test 2, while in the other tests a few modes were missing. The results of both system identification techniques are coincident. As long both techniques are completely independent and based on different numerical approaches, the coincident results confirm that the identified modes corresponded to actual vibration modes and not only to numerical artefacts.

The deficient results in MAC obtained when Test 3 is compared to the others may be interpreted as a dissimilar mode identification in Test 3. This can be attributed to the difference in the time frame considered for performing the data recording. Test 1 and 2 were performed one hour earlier than Test 3 (see Table 3) and that may implied differences in source of excitation.

The fundamental hypothesis that inspired this study was clearly confirmed. Numerical models must be calibrated using experimental measurements if an accurate representation of the actual structures behaviour is desired. Otherwise, the numerical model is only representative of itself and no trustful structural diagnosis or prediction can be extracted from it.

The low dispersion observed in the detected frequencies and mode shapes demonstrates an accurate identification of modal parameters. These parameters will be considered as target values for model calibration in future works. It is expected that the results of this last stage of the study (model updating) can be presented in the conference.

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