SEISMIC RESPONSE OF BRIDGES WITH INTEGRAL DECK-ABUTMENTS: AMBIENT VIBRATION TESTING

J.C. Carvajal[1], C.E. Ventura[2], S. Huffman[3] and F. Yao[4]

[1]*Research Assistant,* [2]*Professor,* [4]*Lab. Manager, Dept. of Civil Engineering, University of British Columbia, Vancouver, Canada*
[3]*Bridge Seismic Engineer, Ministry of Transportation and Infrastructure - TRAN, Victoria, Canada*
Email: jccu@civil.ubc.ca, ventura@civil.ubc.ca, Sharlie.Huffman@gov.bc.ca

ABSTRACT:

Integral abutment bridges are single or multiple span structures consisting of a continuous concrete deck integrated with abutments and a flexible support system. These structures have become very popular due to the elimination of costly and maintenance prone expansion joints and bearings. However, the exclusion of joints causes the superstructure to interact more closely with the soil embankments, making the evaluation of vibration properties and seismic demands a process that involves many uncertainties. The purpose of this paper is to contribute to the better understanding of this type of bridges by investigating in detail their vibration characteristics at low levels of excitation. The paper presents the identification of modal vibration properties of six bridges with integral deck-abutments using ambient vibration tests. Results indicate that the motion of the embankments causes the superstructures of some single-span bridges to deflect with an operational mode similar to the first vertical natural mode of vibration. This behavior suggests that the seismic response of the bridges in the vertical direction may be strongly influenced by the seismic response of the embankments and the abutments. Steel bridges have developed thick cracks at the end of the approach slabs in the embankments. These bridges have the lowest fundamental frequencies of all the tested bridges. The presence of these cracks suggests that the abutment foundation may be too flexible for this type of superstructures. The identification of predominant frequencies of vibration of the embankments is done with a proposed technique named "cross response spectrum". This technique is useful when the identification of predominant frequencies can not be easily determined by more traditional identification techniques such as the power spectral density and the H/V ratio.

KEYWORDS: natural frequency, mode shape, modal damping, response spectra.

1. INTRODUCTION

Seismic assessment of bridges requires an educated prediction of their seismic capacity and demand under various earthquake scenarios. In such an evaluation, there are many sources of uncertainty that engineers have to deal with. These uncertainties are due to: a) the selected loads and material properties, and b) the prediction of the structural response. One of the sources of uncertainty in predicting the structural response is the soil-foundation-structure interaction (SFSI). SFSI can significantly change the response of the structure. For instance, the foundation response, which determines the input to the structure, can differ from that of the site, which in many cases is assumed as the input to the bridge. Another source of uncertainty in the structural response is the soil-abutment-structure interaction (SASI). SASI is usually modeled by replacing the backfill material with equivalent springs and dampers connected to the wall abutments. This approach simplifies the problem; however, it ignores the time-dependent variation of inertial forces induced by the backfill against the abutments during a seismic event. SFSI and SASI can significantly change the actual vibration properties of the bridge in comparison to the predicted with the commonly fixed-base assumption.

In this regard, The University of British Columbia (UBC) and the Ministry of Transportation and Infrastructure (TRAN) initiated in 2007 a joint study to evaluate the seismic response of bridges with integral deck-abutments in the province. The study employs 3D finite element modeling of the structures and their soil embankments to evaluate the effects of SFSI and SASI on these types of bridges for different scenarios of seismic hazard. The first part of this study consists of field experimental work to create a database of in-situ vibration properties of typical integral abutment bridges in the province. This database will be used to validate and update the finite element models of the bridges and their embankments. Ten integral abutment bridges have been tested up until now using ambient vibration equipment. This paper presents and discusses the database of six of these bridges.

2. DESCRIPTION OF BRIDGES

The structures selected for this paper are single and multi span concrete or steel bridges located along the highway 19 in Vancouver Island, British Columbia, Canada. These bridges were built between 1999 and 2001 and each one has a different configuration of the superstructure and the abutment foundation. All bridges have a 0.3m thick cast-in-place reinforced concrete deck with 0.8m high reinforced concrete parapets. Table 1 includes a general description of the bridges.

Table 1. Integral deck-abutment bridges tested in British Columbia

Bridge	Spans		Deck	Clearance	Superstructure		
	no.	length (m)	width (m)	(m)	material	description	depth (m)
Royston Road	1	43.5	11.0	6.0	concrete	two post-tensioned cast-in-place girders	2.2 to 0.8
Forbidden Plateau	1	42.0	12.8	5.5	steel	two single-cell trapezoidal box girders	1.4
Duncan Bay	1	37.0	13.4	5.7	concrete	one cast-in-place box girder	2.0 to 1.5
Millar Creek	1	26.8	23.2	6.2	concrete	six prestressed precast I girders	1.7
Browns River	3	15 - 60 - 15	23.2	11.3	steel	six I girders	2.1
Black Creek	3	20 - 32 - 20	23.2	8.2	concrete	six prestressed precast I girders	1.7

ROYSTON ROAD BRIDGE. The superstructure of this single-span overpass bridge consists of two post-tensioned cast-in-place concrete girders of variable depth along the span (Figure 1). The first 10m of the superstructure at each end is a concrete box formed by the post-tensioned girders, a 0.4m thick reinforced concrete slab and the deck. The separation of the girders is 6.0m and the width of each one is 0.9m. The concrete footing of the post-tensioned abutments is supported directly on the bedrock. The total length of the bridge, including the two 7.0m long approach slabs integrated with the U-shaped abutments, is 57.5m.

Figure 1. Royston Road Bridge

FORBIDDEN PLATEAU ROAD BRIDGE. This single-span overpass bridge has two single-cell trapezoidal steel beams of constant depth along the span (Figure 2). The separation of the beams is 6.6m and the width of each one is 2.4m at top and 1.8m at bottom. The abutments are supported on concrete filled steel pipe piles that extend to the bedrock. The 5m long approach slabs are supported on the embankments and with one end connected to the abutments. One of the embankments has a thick crack located approximately at the end of the approach slab. The deck shows a regular pattern of thin cracks perpendicular to longitudinal axis of the bridge.

Figure 2. Forbidden Plateau Road Bridge

DUNCAN BAY MAIN BRIDGE. The superstructure of this single-span overpass bridge consists of a cast-in-place concrete box girder of variable depth along the span (Figure 3). The width and thickness of the concrete box are 3.6m and 0.3m, respectively. The concrete footing of the U-shaped abutments is securely anchored to the bedrock. The total length of the bridge, including the two 7.0m long approach slabs integrated with the abutments, is 51.0m. The deck shows an irregular pattern of thin cracks.

Figure 3. Duncan Bay Main Bridge

MILLAR CREEK BRIDGE. This single-span bridge consists of six prestressed precast concrete I girders of 1.7m depth. The girders are spaced at 4.0m apart (Figure 4). The concrete footing of the abutments is supported directly on bedrock. The 6m long approach slabs are supported on the embankments and with one end connected to the abutments. The deck shows a regular pattern of thin cracks oriented at 45° with respect to the longitudinal axis of the bridge and mainly concentrated at the abutments area.

Figure 4. Millar Creek Bridge

BROWNS RIVER BRIDGE. The superstructure of this multi-span bridge consists of six steel I girders of 2.1m depth. The girders are spaced at 4.0m apart (Figure 5). The abutments are supported on concrete filled steel pipe piles anchored to bedrock. The spread footings of the 0.9m diameter concrete piers are supported directly on bedrock. The 6m long approach slabs are supported on the embankments and with one end connected to the abutments. Both embankments have thick cracks located at the end of the approach slabs.

Figure 5. Browns River Bridge

BLACK CREEK BRIDGE. This multi-span bridge consists of six prestressed precast concrete I girders of 1.7m depth (Figure 6). The abutments are supported on concrete filled steel pipe piles that extend to the bedrock. The 0.8m diameter concrete filled steel pipe piles are embedded into the bedrock. The 6m long approach slabs are supported on the embankments and with one end connected to the abutments. One of the embankments has a thin crack located at the end of the approach slab. The deck shows a regular pattern of thin cracks oriented at 45° with respect to the longitudinal axis of the bridge and mainly concentrated at the pile bend area.

Figure 6. Black Creek Bridge

3. EXPERIMENTAL FIELD TESTS

Ambient Vibration Testing (AVT) is an accurate and cost-effective technique for obtaining modal parameters of large structures [1, 2]. The technique consists of measuring the structure response to ambient forces such as wind, traffic, human activities, etc. Data is then processed using algorithms of Modal Operational Analysis [3, 4], also known as Output Only Modal Analysis, to obtain the modal parameters of the structure. The main advantages of AVT are: a) Equipment for exciting the structure is nod needed, b) Testing does not interfere with the normal operation of the structure, and c) The measured response is representative of the real operating conditions of the structure [5]. Data quality in AVT depends significantly on the recording system. The equipment generally consists of high sensitive accelerometers (or other types of sensors), cables, and a multi-channel data acquisition system. Some of the inconveniences of the equipment are related to cable handling, sensor balancing, signal clipping, and power supply.

The ambient vibration system of the Earthquake Engineering Research Facility (EERF) at UBC consists of three wireless Pinocchio WL380 geophone-based units capable of handling long measurements (up to 12hr) at high sampling rates (up to 500sps) without external power supply. Each unit has a GPS antenna for acquiring precise time synchronization and two sets of tri-axial geophones to measure low and high amplitude motion. The geophones have natural frequency of 4.5Hz and 56% damping. The units are programmed using a laptop and secure digital cards. At the completion of the testing, the cards are removed from the units and data is downloaded to a laptop.

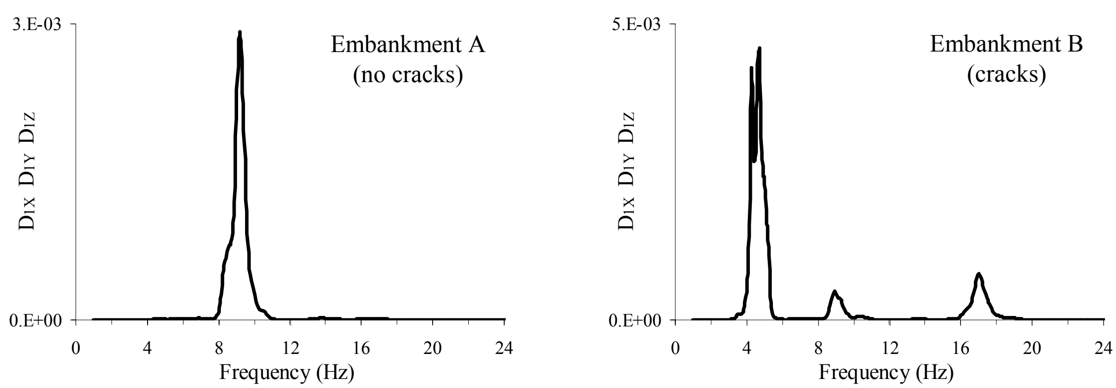
The AVT campaign of ten integral abutment bridges in British Columbia was performed in July 2007 by two members of the EERF-UBC. Each bridge was tested along the two sides of the deck. The distribution of the testing locations had the objective of identifying up to the third or fourth vertical mode of the superstructures. The number of testing locations depended on the length of each bridge bridge. Each embankment was tested at five locations along one side of the road, covering a distance of 100m from the abutment. The units were programmed to get time histories at each measured location of five-minute duration for single-span bridges or ten-minute duration for multi-span bridges. The sampling rate was 500sps. One unit was placed on the deck (reference sensor) while the other two (roving sensors) were placed at the testing locations. The time required for testing a bridge varied from 4 to 6 hours.

4. DATA ANALYSIS, RESULTS AND DISCUSSION

4.1. Cross Response Spectrum Technique

Identification of Predominant Frequencies of Vibration (PFV) of soil embankments, using ambient vibration time histories, is done with a proposed technique based on the displacement response spectrum (D). The technique evaluates D at each testing location for each direction of motion: D_X , D_Y and D_Z and normalizes the spectra to 1: D_{1X} , D_{1Y} and D_{1Z}. These three normalized spectra are multiplied among them to obtain a cross response spectrum at each location, D_{1X} D_{1Y} D_{1Z}. PFV are identified from a global cross response spectrum that includes all the individual cross response spectra of the abutment. The reason of obtaining a cross response spectrum to identify PFV is based on the assumption that the motion of soil embankments is caused by surface waves, which have a 3D motion path [6]. This technique has been useful when the identification of PFV is not clear with techniques such as power spectral densities [7] and the H/V ratio [8].

Figure 7 shows, for example, the global cross response spectra of the two embankments of the Forbidden Plateau Bridge. The frequency step of the spectra is 0.1Hz and damping is 2%. Figure 7a clearly indicates that the PFV of the embankment A is $F_A \approx 9.2$Hz. A visual inspection of this embankment did not find any anomalies such as crack or settlements. Figure 7b, on the other hand, indicates that the PFV of the embankment B is $F_B \approx 4.7$Hz. This embankment has a thick crack at the end of the approach slab. Figure 7b also indicates that other PFV of this embankment are $F_B \approx 9.2$ and 17.0 Hz. The cross response spectrum technique was efficient to identify anomalies associated with cracks in the embankments.



(a) (b)

Figure 7. PFV of the embankments of the Forbidden Plateau Road Bridge

4.2. Enhanced Frequency Domain Decomposition

Identification of modal vibration properties of the superstructures can be performed by the Enhanced Frequency Domain Decomposition (EFDD) technique, implemented in the commercial computer program ARTeMIS [9]. The technique is based on the approximate decomposition of the system response into a set of independent Single Degree of Freedom (SDOF) systems. Modal Parameters are estimated by performing a Singular Value Decomposition (SVD) of the Power Spectral Density (PSD) matrix. Identification of SDOF systems is based on the SVD plot and the peak-picking technique. The identified SDOF-PSD functions around the peaks of resonance are taken back to the time domain using the Inverse Discrete Fourier Transform. Then the natural frequencies are obtained by determining the number of zero-crossing as a function of time, and the damping by the logarithmic decrement of the corresponding SDOF normalized auto correlation function [10].

Figures 8 to 9c show, for example, the identification of the first three natural modes of vibration of the Royston Road Bridge using the SVD plot and the peak-picking technique. The singular values (frequencies) at $F_1 = 4.8$Hz, $F_2 = 7.0$Hz and $F_3 = 8.9$Hz (Figure 8) correspond to the first vertical vector (Mode 1: Figure 9a), first torsional vector (Mode 2: Figure 9b), and second vertical vector (Mode 3: Figure 9c), respectively. Figures 9d to 9e show five more modes of vibration identified from the SVD plot (not shown in Figure 8).

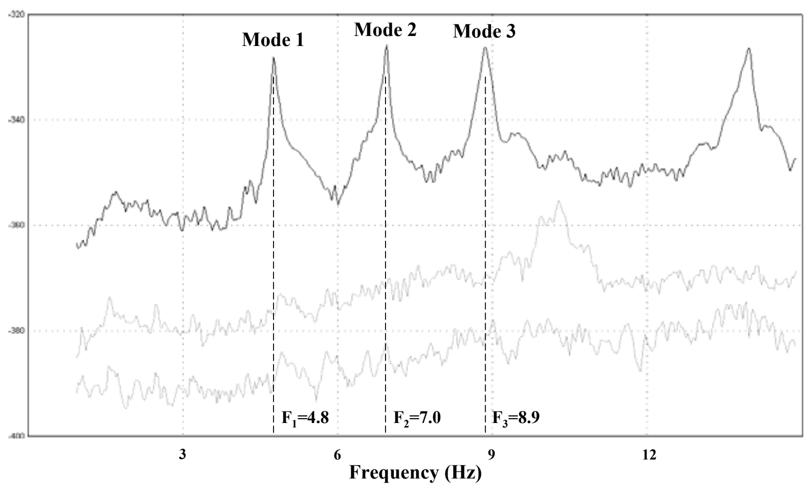
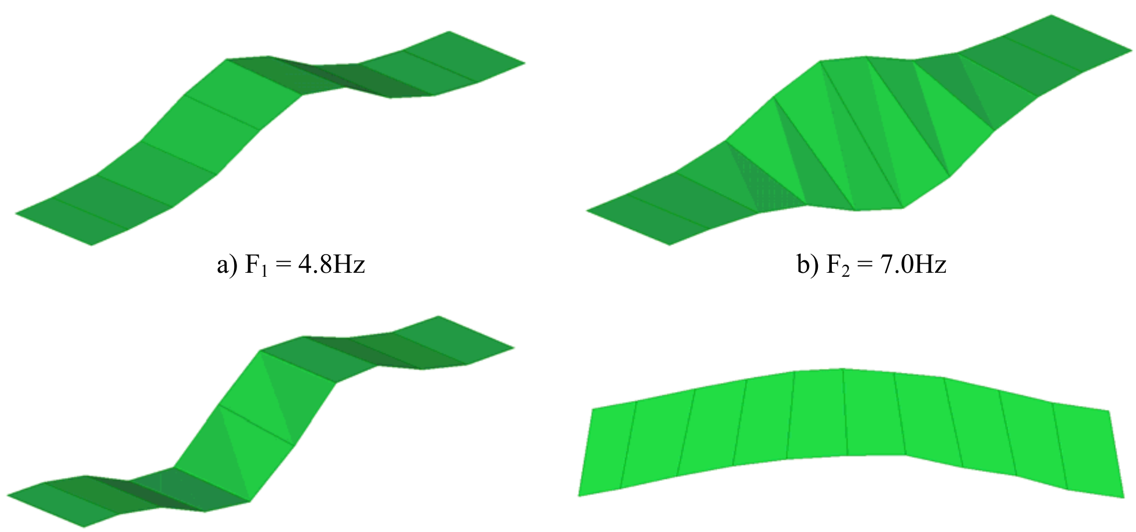


Figure 8. Identification of natural modes using the SVD plot of the Royston Road Bridge



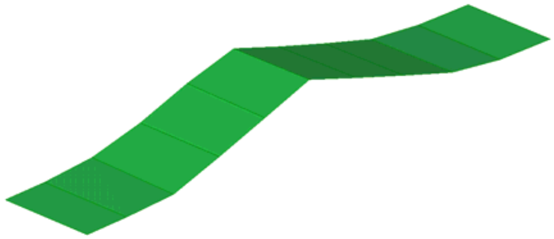
a) $F_1 = 4.8$Hz

b) $F_2 = 7.0$Hz

c) $F_3 = 8.9$Hz

d) $F_4 = 10.4$Hz (plan view)

Figure 9. Identification of mode shapes of the Royston Road Bridge using field ambient vibration data



e) F_5 = 14.0Hz

f) F_6 = 15.2Hz

g) F_7 = 23.9Hz

h) F_9 = 34.3Hz

Figure 9(cont'). Identification of mode shapes of the Royston Road Bridge using field ambient vibration data

4.3. Results and discussion

ROYSTON ROAD BRIDGE. Modal vibration properties of this single-span concrete bridge are presented in Table 2. The PFV of the embankments A and B is approximately F ≈ 14.0Hz. Figures 9a and 9e show two almost identical mode shapes associated with the frequencies F_1 = 4.8Hz and F_5 = 14.0Hz. This unexpected mode shape at 14.0Hz coincides with the PFV of the approaches. A qualitative analysis suggests that the buckling resistance of this superstructure is very low due to the high slenderness of the arch type girders and the low level of vertical restriction provided by the deck. This causes the girders to deflect in the vertical direction when an axial compressive force is applied at the flexible abutments. The motion of both approaches is not perfectly-in-phase, which induces an axial compressive force along the bridge. This explains the nature of the unexpected operational mode of the superstructure at the PFV of the embankments. The fundamental frequency of vibration of the superstructure is F_1 = 4.8Hz which corresponds to the first vertical mode. Modal damping of this bridge varies from 0.4 to 1.4% with mean value $\xi_m \approx 0.76\%$.

FORBIDDEN PLATEAU ROAD BRIDGE. Modal vibration properties of this single-span steel bridge are presented in Table 3. Only a few natural modes were detected in this type of superstructure using AVT. The second torsional mode at F_4 = 10.7Hz is coupled with an operational mode of low amplitude similar to the first vertical natural mode. As previously discussed, this may be associated with the buckling-like effect caused by motion of the embankments. The PFV of the embankment B ($F_B \approx 4.7$Hz) is approximately half of the PFV of the embankment A ($F_A \approx 9.2$Hz). As mentioned before, Embankment B has a thick crack at the end of the approach slab. This indicates that the amplitude and frequency of rotation of the two abutments are different under normal operational conditions. The non-symmetry of the dynamic response of the abutments can induce significant forces in the superstructure during a seismic event. The fundamental frequency of vibration of the superstructure is F_1 = 3.6Hz which corresponds to the first vertical mode. Modal damping of this bridge varies from 0.6% to 1.2% with mean value $\xi_m \approx 1.0\%$.

DUNCAN BAY MAIN BRIDGE. Modal vibration properties of this single-span concrete bridge are presented in Table 4. The PFV of the embankments is 9.0Hz, approximately, and there is no evidence of cracking in them. The first transverse mode at F_2 = 10.4Hz has a torsional component of very low amplitude, typical of box-shaped superstructures. Analysis of ambient vibration data did not detect any operational modes in the superstructures associated with the motion of the embankments. The fundamental frequency of vibration of the superstructure is F_1 = 5.5Hz which corresponds to the first vertical mode. Modal damping of this bridge varies from 0.3% to 0.8% with mean value $\xi_m \approx 0.62\%$.

Table 2. Modal parameters of Royston R. Bridge

	order	Mode characteristic	Freq. Hz	ξ %
Bridge	1	First Vertical	4.8	0.8
	2	First Torsional	7.0	0.6
	3	Second Vertical	8.9	1.1
	4	First Transverse	10.4	1.4
	5	operational	14.0	0.6
	6	Third Vertical	15.2	0.8
	7	Fourth Vertical	23.9	0.5
	8	Third Torsional	27.7	0.4
	9	Fourth Torsional	34.3	0.6
Embankment	A	13.8 Hz		
	B	14.0 Hz		

Table 3. Modal parameters of Forbidden P. Bridge

	order	Mode characteristic	Freq. Hz	ξ %
Bridge	1	First Vertical	3.6	1.0
	2	First Torsional	5.2	0.6
	3	Second Vertical	8.7	1.2
	4	Second Torsional	10.7	1.2
Embankment	A	9.2 Hz		
	B	4.7, 9.2, 17.0 Hz	*cracks*	

MILLAR CREEK BRIDGE. Modal vibration properties of this single-span concrete bridge are presented in Table 5. Natural mode shapes of this type of superstructure are mainly formed by a combination of the vertical deflection shapes of the six girders. Then the superstructure has mode shapes in the longitudinal (called vertical* in Table 5) and transverse (called torsional* in Table 5) direction of the bridge in a similar form to plate modes. The proper identification of mode shapes in this type of superstructures requires testing locations along six longitudinal axes of the bridge. Unfortunately, this was not possible since the bridge was in operation. Table 5 includes the identification of frequencies and modal damping based on measurements along two longitudinal axes of the bridge. The PFV of the embankments is 20.0Hz, approximately. The cross response spectrum technique also identified a PFV of 10Hz in the embankments which can be associated with the fundamental frequency of the bridge which acts as a source of surface waves. The lowest frequencies of vibration of the superstructure are F_1 = 9.5Hz and F_2 = 9.8Hz which correspond to the first torsional and vertical natural modes, respectively. Modal damping for this bridge varies from 0.4% to 2.0% with mean value $\xi_m \approx 1.0\%$.

Table 4. Modal param. of Duncan Bay M. Bridge

	order	Mode characteristic	Freq. Hz	ξ %
Bridge	1	First Vertical	5.5	0.7
	2	First Tranverse	10.4	0.8
	3	Second Vertical	12.0	0.8
	4	First Torsional	13.7	0.5
	5	Second Torsional	20.4	0.8
	6	Fourth Vertical	29.2	0.3
	7	Fourth Torsional	32.6	0.4
Embankment	A	9.1 Hz		
	B	9.0 Hz		

Table 5. Modal parameters of Millar C. Bridge

	order	Mode characteristic	Freq. Hz	ξ %
Bridge	1	First Torsional	9.5	1.4
	2	First Vertical	9.8	0.8
	3	vertical*	10.7	1.3
	4	vertical*	12.4	2.0
	5	torsional*	15.3	2.0
	6	torsional*	21.5	0.5
	7	torsional*	23.1	0.6
	8	torsional*	27.1	0.5
	9	vertical*	28.3	0.7
	10	torsional*	31.1	0.4
	11	torsional*	34.1	0.7
Embankment	A	10.0, 20.0 Hz		
	B	9.7, 20.9 Hz		

BROWNS RIVER BRIDGE. Modal vibration properties of this multi-span steel bridge are presented in Table 6. The PFV of the embankments are $F_A \approx$ 16.7Hz and $F_B \approx$ 19.0Hz. A PFV of 2.5Hz, approximately, was also detected in both embankments, which have thick cracks at the end of the approach slabs. This low frequency is associated with rotation of the abutments and sliding of the approach slabs connected to them. The lowest frequencies of vibration of the superstructure are F_1 = 2.2Hz and F_2 = 2.3Hz which correspond to the first vertical and torsional modes, respectively. This small difference of 0.1Hz between the two lowest modes causes the superstructure to vibrate as a modulated amplitude oscillator in the vertical direction during the passage of heavy traffic. Modal damping for this bridge varies from 0.2% to 1.2% with mean value $\xi_m \approx 0.61\%$.

BLACK CREEK BRIDGE. Modal vibration properties of this multi-span concrete bridge are presented in Table 7. The PFV of the embankments are $F_A \approx 11.6$Hz and $F_B = 12.2$Hz. A PFV of 3.0Hz was also detected in the embankment B, which has a thin crack at the end of the approach slab. The lowest frequencies of vibration of the superstructure are $F_1 = 6.1$Hz and $F_2 = 6.6$Hz which correspond to the first vertical and torsional modes, respectively. Modal damping of this bridge varies from 0.5% to 0.9% with mean value $\xi_m \approx 0.75\%$.

Table 6. Modal param. of Browns River Bridge

	order	Mode characteristic	Freq. Hz	ξ %
Bridge	1	First Vertical	2.2	0.4
	2	First Torsional	2.3	0.5
	3	vertical*	2.9	0.6
	4	torsional*	5.1	1.2
	5	Second Torsional	5.9	0.5
	6	Second Vertical	6.1	0.6
	7	torsional*	7.6	0.6
	8	vertical*	9.4	1.1
	9	vertical*	10.9	0.6
	10	Third Vertical	11.3	0.2
	11	torsional*	16.3	0.4
Embankment	A	2.3, 9.7, 16.7 Hz	*cracks*	
	B	2.6, 17.3, 19.0 Hz	*cracks*	

Table 7. Modal parameters of Black C. Bridge

	order	Mode characteristic	Freq. Hz	ξ %
Bridge	1	First Vertical	6.1	0.9
	2	First Torsional	6.6	0.7
	3	vertical*	7.1	0.8
	4	torsional*	8.5	0.8
	5	vertical*	11.4	0.9
	6	torsional*	14.8	0.8
	7	vertical*	15.7	0.5
	8	vertical*	17.4	0.8
	9	torsional*	18.8	0.6
Embankment	A	10.3, 11.6 Hz		
	B	3.0, 12.3, 16.9 Hz	*cracks*	

5. SUMMARY AND CONCLUSIONS

Ten integral deck-abutments bridges have been tested up until now in British Columbia, Canada, using ambient vibration equipment. The objective of the testing campaign is to create a data base of in-situ modal vibration properties to calibrate and update 3D finite element models of the structures and their embankments. This paper presented and discussed the database of six of these bridges.

Analysis of ambient vibration data indicates that the motion of the embankments causes the superstructures of some single-span bridges to deflect with an operational mode similar to the first vertical natural mode of vibration. This behavior is more noticeable in the arch bridge with abutment foundation simply supported on the bedrock (Royston R. Bridge). The detection of this operational mode using low amplitude motion data suggests that the dynamic response of this type of bridge can be strongly amplified in the vertical direction by the dynamic response of the embankments and the abutments during a seismic event. This type of operational mode was not detected in bridges consisting of a concrete box girder superstructure with U-shape abutments securely anchored to the bedrock (Duncan B.M. Bridge).

Severe cracking was observed in the embankments of steel bridges. The cracks were located at the end of the approach slabs. These steel bridges have lower fundamental frequencies ($F_1 < 3.6$Hz) than the concrete bridges ($F_1 > 4.8$Hz). The cracking may be caused by large rotation of the abutments due to large vertical displacements of the flexible superstructures during the passage of heavy traffic. The rotation could also be due to axial displacements in the steel superstructures due to temperature changes. The presence of these cracks suggests that the abutment foundation (one row of concrete filled steel pipe piles) may be too flexible for this type of superstructures. The effects of the flexibility of the abutment foundation on the seismic demands of the superstructure (SASI) and the stability of the embankments (settlements) during a seismic event will be studied with the calibrated 3D finite elements models.

The predominant frequencies of vibration (PFV) of the embankments detected with the cross response spectrum technique varied from 9.0Hz to 20Hz. Low PFV (< 4.7Hz) were also detected in the embankments with cracks located at the end of the approach slabs. This indicates that the rotation of the abutments and the sliding of approach slabs connected to them are acting as a source of surface waves with frequencies that approximately coincides with the fundamental frequencies of the superstructures.

The mean value of the modal damping of the superstructures obtained with the enhanced frequency domain decomposition technique varied from 0.6% to 1%.

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