1. Introduction

Structural health monitoring is regarded as the main tool in assessing the functionality of existing structures. The importance of these techniques emerges by considering that failure of an infrastructure results in catastrophic loss. Existing civil structures deteriorate by aging and under different loading conditions imposed from impacts from accidents or natural phenomena such as earthquakes, typhoons, flood and etc. Therefore, it is imperative to investigate the safety of continuing to use these structures, especially after occurring major loads on them from these phenomena.

Ambient Vibration Tests (AVTs) are widely used for system identification and damage detection in the bridge structures [1]. Kaya et al [2] and Kohler et al [3] used this technique for damage detection and modal analysis of building structures before, respectively. The current study focuses on a series of Ambient Vibration Test which were conducted at the Huntington Bridge, located in Abbotsford, BC, Canada in order to determine the modal frequencies and the mode shapes of the structure. The sensors were placed on predetermined locations on the bridge deck. A series of vibration tests were then carried out using velocity sensors. The main purposes of the AVTs were to determine the fundamental frequency and corresponding mode-shape of the deck in the current condition.

Huntington Bridge was constructed in mid 1960s in the south side of Abbotsford, BC. This bridge is one of the major connecting highway bridges to connect Abbotsford to the US-Canada border. This structure is located in the south east part of Abbotsford and connects the highway to Huntington, Abbotsford, BC. A total of 6 lanes pass on top of the bridge and a total of 4 lanes pass from under the bridge. This bridge is structurally composed of 6 separate bridges which will be discussed in later chapters. The structure is resting on elastomers on each end of the spans and one of the spans is accommodates the soil shoulder of the road while the other two accommodate the four under-passing lanes. Figure 1 shows the satellite views of the Huntington Bridge and Figure 2 illustrates the north and south views of the bridge.

Fig. 1 - Satellite views of the Huntington Bridge

(a) (b)

Fig. 2 - General view of the bridge: a) North view; b) South view

This structure was previously hit by a passing truck underneath of the bridge. The middle north part of the bridge was impacted by the crown of a loading truck hitting 2-3 girders. One of these girders was damaged heavily which resulted in closing the bridge and retrofitting the girders. This girder was repaired by attaching the pre-stressed strands together with extensions and filling up the cavity with new grout. Finally, an extra steel I-beam was located besides the damaged girder which was attached with several connection keys to the damaged girder. This bridge is designed based on AASHO design code 1953 ED H20 Loading. The repair was later done based on Canadian Highway Bridge Design Code CSA-S6-06.

The structure is made by pre-stressed concrete girders with 8-days strength of 5500 lb.in2 and uncoated strands with ultimate strength of 20000 lb per strand. The service life of this structure for the design was 100 years. The structure of the Huntington bridge is by design composed of 6 separate bridges located side by side in two directions of the horizon plane. The separation of these components can be clearly seen in the Figures.

In this paper a series of studies are performed and targeted on Huntington Bridge, as part of creation of a framework to assess the safety of continuing the operation of such important infrastructures after occurrence of major impacts on them. This structure was damaged previously by an impact on several girders from a truck passing underneath the bridge. A series of Ambient Vibration Tests (AVT) were conducted at the Huntington Bridge in Abbotsford, BC in order to determine the modal frequencies and the mode shapes of the bridge structure. The sensors were placed on predetermined locations according to the test plan, i.e. on deck and on the abutment. The main purposes of the AVTs were to determine the fundamental frequency and corresponding mode-shapes of the bridge in the current condition. This will help in calibrating analytical models to calculate the load redistribution on the bridge after a possible similar damage. Furthermore, a comprehensive practical step-by-step procedure is proposed for safety assessment of a highway bridge damaged due to an earthquake or accident.

The scope of this study is limited as follows:

1. This study has been conducted on a reinforced concrete bridge with precast pretensioned girders and supporting pairs, specifically on Huntington Bridge. So, the framework of this project is limited to this typical bridge. Any other types of the bridges, such as suspension or cable-state should be studied as separate project;
2. The proposed procedure has been designed to assess the safety of the bridge which was damaged in an accident caused by over-height truck. The Huntington Bridge with the same type of damage also has been selected as a case study. So, the proposed method does not applicable to other types of damage;
3. The data from Ambient Vibration Tests in the context of operational modal analysis is considered. Accordingly, the structure is assumed to behave linear elastic, under ambient vibrations. Non-linearity or inelasticity are not in the scope of this report;
4. Moreover, the damage is considered as a linear damage. The linear damage is defined as a damage that does not change the behavior of a linear structure to nonlinear behavior. The extent of the damage is considered to be from cracks up to sever damages without failure: the structure is still operational and no failure has happened;
5. The environmental effects such as temperature, moisture, soil-structure interaction, and changes in the statistical characteristics of input excitations are excluded.

2. Ambient Vibration Tests

AVTs were conducted on the Bridge in order to determine the dynamic modal properties (modal frequencies and mode shapes) of the structure. The testing program consisted of five measurement setups on the deck. The test information for each setup, and the average temperature at the time of the tests are presented in Table 1. The setup 1 was repeated to provide reliable and sufficient data for modal analysis. So, the setups 2, 3, and 4 were used for analyzing the collected data in order to determine the dynamic parameters of the Bridge. [4].

Table 1 - Test information for each setup

	Setup 1	Setup 2	Setup 3	Setup 4	Setup 5
Date of the Test	14 May 2018				
Time of the Test	22:13:41	22:44:15	23:13:20	23:42:30	00:13:17 (+1 Day)
Number of the Sensors	9	9	9	9	9
Recording Duration (min.)	26	26	26	26	26
Temperature (℃)	20	20	18	18	17

The structure composition can therefore be depicted on the bridge as 2 rows of bridges on which three separate spans are located as shown in Figure 3. It should be noted that although the adjacent parts of the bridges are separate, these gaps are filled with dirt and other materials during the years that this bridge is in operation. These solid filaments can cause some force interaction between the adjacent separate parts of the bridge. Layout of the sensors on the bridge deck is shown in Figure 3.

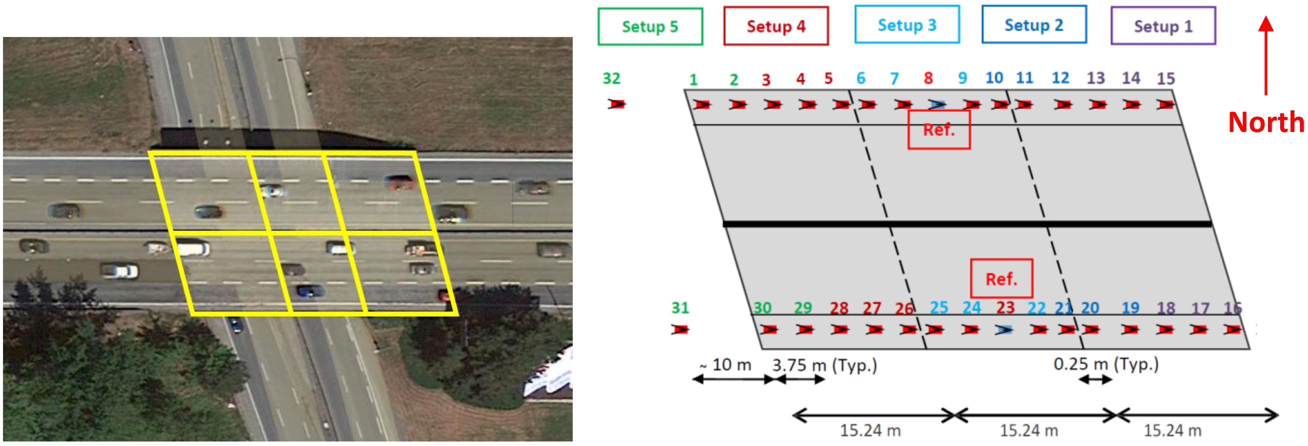


Fig. 3 - Bridge components highlighted as 6 separate decks and Layout of the sensors on the bridge deck

Tromino® velocity/acceleration sensors [5] were used to carry out the AVTs. The collected records were time synchronized with a radio antenna and amplifier in each sensor. This allowed the synchronization of the recordings both within each measurement setup and between setups. The Tromino sensors are suitable for high-resolution ambient vibration tests as they are fully portable, wireless, compact, and light instruments. Each sensor is equipped with two sets of three orthogonal high-resolution electrodynamics sensors (high gain and low gain velocity meter) and one set of three orthogonal digital accelerometers with a frequency range of 0.1 to 300 Hz. For these tests the high-gain velocity data was used for the modal identification process. The sampling frequency of the recordings at each setup was 128 samples per second (sps), and the total recording duration for each setup was about 26 minutes. This testing approach allows to capture the most important vibration modes up to a frequency of about 64Hz. The North component of each sensor was oriented to the North direction of the site for all setups. One stationary reference sensor was used for all setups. Table 2 presents the location of the sensors on the bridge deck.

Table 2 - Location of the sensors on the bridge deck (All Setups)

Setup1		Setup2		Setup3		Setup4		Setup5	
Sensor#	**Location**	**Sensor#**	**Location**	**Sensor#**	**Location**	**Sensor#**	**Location**	**Sensor#**	**Location**
1	8	1	8	1	8	1	8	1	8
2	23	2	23	2	23	2	23	2	23
3	18	3	21	3	25	3	26	3	29
4	17	4	20	4	24	4	27	4	30
5	16	5	19	5	22	5	28	5	31
6	15	6	12	6	9	6	3	6	32
7	14	7	11	7	7	7	4	7	1
8	13	8	10	8	6	8	5	8	2

3. Data Processing and Modal Analysis

The computer program ARTeMIS version 4 [6], was used to perform the modal identification of the structure. The software allows the user to develop a 3D model of the structure and test points; the resulting mode shapes are displayed using this geometry. Two different, complementary techniques are usually used for modal identification [7]: Enhanced Frequency Domain Decomposition (EFDD) and Stochastic Subspace Identification (SSI). These two modal identification techniques are used to cross-validate the results. The joint analysis of the signals measured in various strategic points of the structure makes it possible to identify the modal configurations and the corresponding natural frequencies.

The FDD technique is a frequency domain method, and the procedure consists of decomposing the system output into a set of systems of a specific degree of freedom, which are independent for each mode. The singular values are estimated from the spectral density of the specific degree of freedom system and the configuration of the modes is estimated from the singular vectors by selecting the highest peaks of the responses.

Ambient vibration data recorded on and off the Huntington Bridge contains both noise and the response of the Huntington Bridge under ambient vibrations. The noise component of the recorded data is mainly due to traffic inconsistency, mechanical imperfections in the sensors, instrument noise, installation, and other aspects in the sensor such as digitalization. The noise components of the vibration data, by its nature, usually appear as a random phenomenon in the data; however, the response of the Huntington Bridge is not random, but consistent at certain frequencies due to resonance effects of the Huntington Bridge to environmental excitations. Removing the noise components from the data is generally achieved by using signal processing tools such as decimation, filtering, and data averaging.

The data collected at all measurement locations was processed and analyzed with ARTeMIS Modal. Several 3D model (for animation purposes) of the Bridge was created using the structural geometry provided by the Ministry of Transportation for each bridge part. The model includes the discretized locations of measurement points on the structure. These models are created by keeping the reference sensors and removing the rest of the sensors that are not located on the part of the bridge being investigated. The blue arrows represent the location and orientation of the reference sensors, while the green and pink arrows represent the location and orientation of the roving sensors. This estimation would be correct since the readings from all of the sensors had been fully synchronized. Therefore, this method would provide a reliable estimation of a mode shape.

In order to identify the modal properties of the Bridge correctly, each part of the bridge was analyzed separately. The whole bridge is also analyzed with all the sensors in place. Figure 4 shows the complete deck model that is including all the 6 parts.

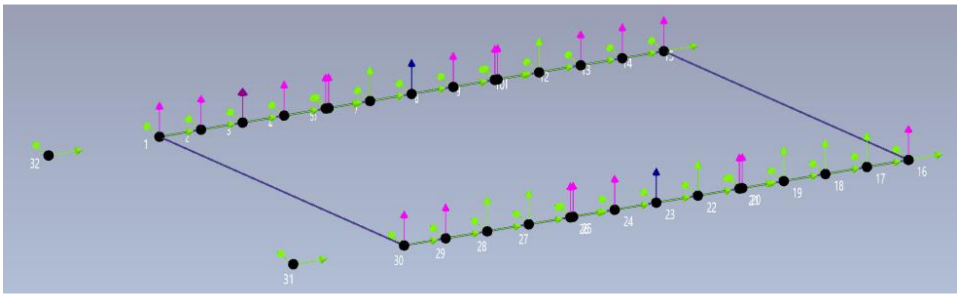


Fig. 4 - 3D ARTeMIS model showing the location of the measurement points of the bridge

4. Test Results

The results of the analysis using the FDD method are shown in Figure 5. This figure shows a plot of the spectral density of the peak singular values of all the data from all the setups as a function of frequency. The peak values in this plot can be associated to dominant frequencies in the datasets, and some of these correspond to modal frequencies.

The results of the modal analysis are presented in this section for each part of the bridge, separately. The first mode shapes of the whole bridge are also shown in this section. It should be noted that due to the separation of the bridge parts, the mode shapes of the bridge as a whole are only approximately correct, and only the first mode shapes which are more stable can be seen.

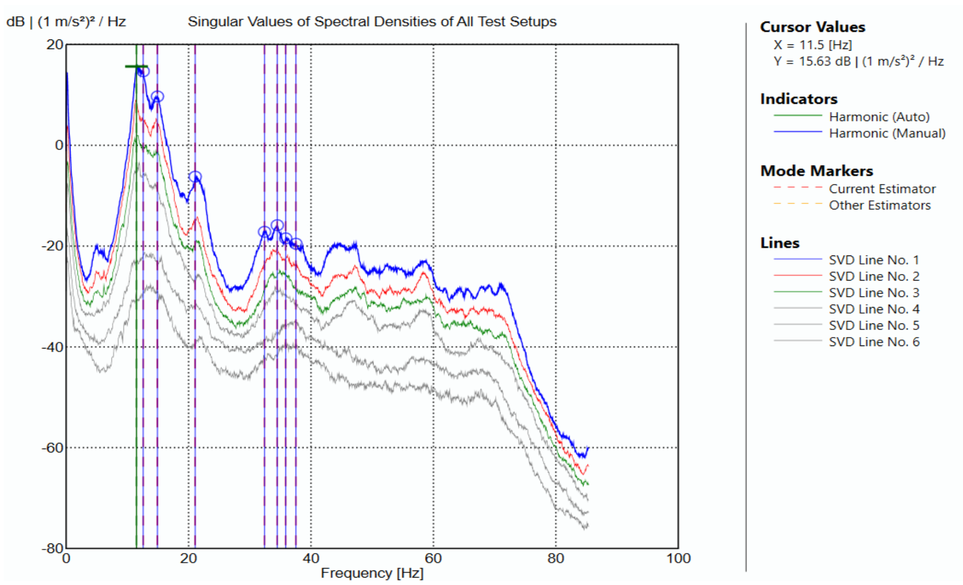


Fig. 5 - Singular Values of Spectral Densities of the bridge with all parts in the frequency range of 0 to 100 Hz using FDD Method

Based on this analysis, the predominant frequency (the first natural frequency) of the structure is estimated to be about 11.5 Hz. The corresponding mode-shape is shown in Figure 6. The higher mode shapes need to be investigated separately for each bridge part. The mode shapes for each part are based on the cross validation with the Finite Element model. Since there is no sensor located on the central part of the bridge (due to limitations on closing the bridge traffic for the test) the torsional mode shapes can only be captured form this cross validation. The mode shapes based on the number of curvatures can be computed only for one side of one of the parts of the bridge.

Table 3 summarizes the modal frequencies extracted from the vibration data for each bridge part using FDD method. As can be seen, the modal frequencies are close to each other due to the fact that each bridge part is built similarly with same characteristics. A Finite Element model is built and updated based on the modal properties of the bridge shown in this table. For this structure, the mode shapes with two curvature points on one side of the bridge will usually match the fifth or sixth mode shape in the table.

Table 3 - Modal frequencies of the bridge from modal analysis estimated by FDD (Hz)

Mode #	North-Left	North-Middle	North-Right	South-Left	South-Middle	South-Right	FE-Model
1	11.417	11.375	11.208	11.5	11.458	11.5	12.40
2	12.417	12.583	12.417	11.875	12.417	12.917	12.81
3	15.167	14.792	14.583	15.042	14.458	14.875	14.74
4	21.375	21.167	21.292	16.75	21.417	21.125	22.5
5	34.875	36.125	34.292	32.417	36.083	34.292	31.98
6	36.833	37.0	37.583	34.5	38.542	36.917	32.78

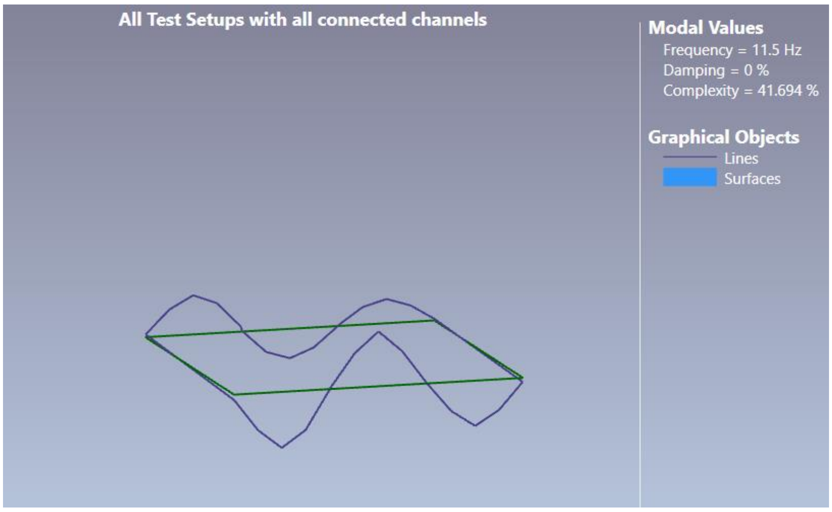


Fig. 6 - 1st mode shape of the bridge as a full structure (11.5 Hz)

5. Finite Element Modeling

Ambient Vibration Tests (AVT) were conducted at the Huntington Bridge in order to determine the dynamic modal properties (modal frequencies and mode shapes) of the bridge. The measurements were used to update a Finite Element model created based on the design of the structure. This will help in identification and cross validation of the acquired mode shapes from the AVT and to simulate damage effects.

Since the structure is composed of 6 similar parts, a finite element model is created to simulate the dynamic behaviour of one part of this bridge. Based on these considerations, the Finite Element model of a part of the bridge is shown in Figure 7.

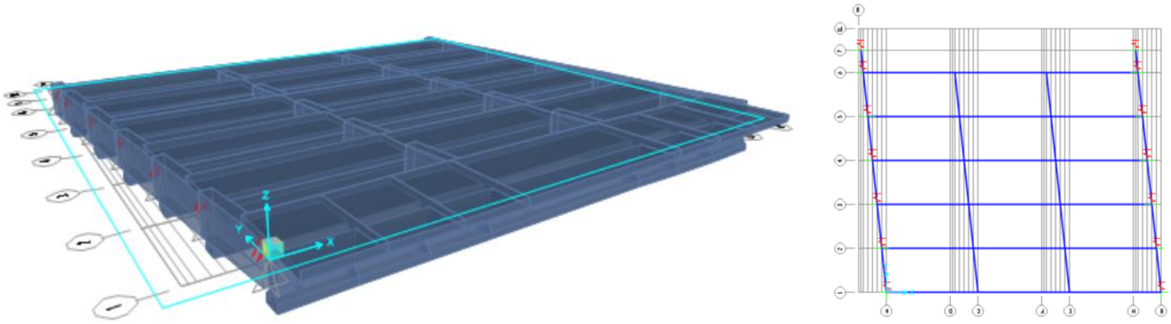


Fig. 7 - Finite Element model of a part of the Huntington Bridge; 3D model (left), plan view (right)

Although the 6 parts of the bridge are separate, the gaps between the adjacent bridge parts are filled with sedimentations and dirt. These filaments result in force transmission between different parts. This effect is modeled through rotational springs on the edges of the structure. These rotational springs are updated based on the modal responses of the bridge. Furthermore, the slab of the bridge is also modelled continuously from one end of the span to the other end. The slab is discontinued in the Y-axis direction, since its contribution to the rotational stiffness in that direction can be ignored as there is no constraints on the edges in X direction. The small beams in the Y direction are continuous to simulate the behaviour of the structure consistently based on the actual bridge structure. The stiffness of the rotational springs equal to 1E9 N.m/rad and this stiffness is updated based on the real modal properties of the bridge which results in an average frequency between fully released and clamped boundary conditions frequencies. The dynamic mass of the strcuture is calcualted based on a concrete slab with thickness of 20.32 cm and asphalt with thickness of 4 cm. The modal values of the FE model are shown in Table 3 and as can be seen they match with the AVT results for different parts of the bridge. The first 6 mode shapes of the FE model and their natural frequencies are shown in Figure 8.

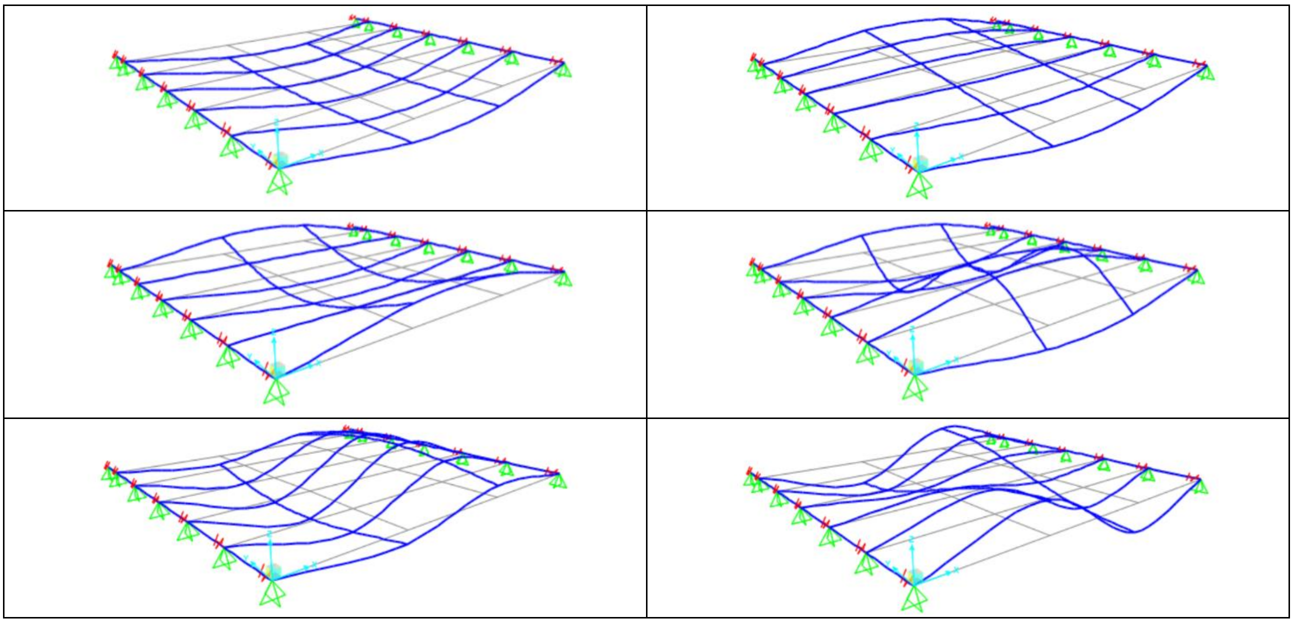


Fig. 8 - Modal parameters of the FE model for the first 6 mode shapes

6. Damage Simulation and Load Redistribution

The results of an investigation on the effect of damage in one girder to the load distribution on all the girders is shown. The stress-strain of a structure is a function of the stiffness matrix and the input load. By assuming a constant input load, the stiffness matrix is the main parameter in redistributing the stress in the structure.

Stiffness matrix itself is a function of the stiffness of each individual element, their connection to other elements and the boundary conditions of the structure. When damage happens in one element of the structure, the stiffness matrix changes and therefore the stress (load) distribution would change in the structure. By having a finite element model from the structure and updating the global model parameters plus local model parameters (representing the damage), the model can be updated to match the AVT test results. The model can then be used to estimate the effect of damage and to compute the redistribution of the loads after the stiffness matrix changes due to the damage. The redistributed loads can be then compared to the load capacity of the elements. This will assure the safety of continuing the use of the bridge or to stop its operation. In this section bridge FE model which was updated based on the AVT results, is used to simulate three levels of damage. The damage location is in the side girder on the North-East part of the bridge. The damage is modeled as the decrease of the height of the girder in the middle section as shown in Figure 9. The reduction of the height of the gidrer varies from 5% (mild damage), to 25% (intermediate damage) and to 60% (severe damage).

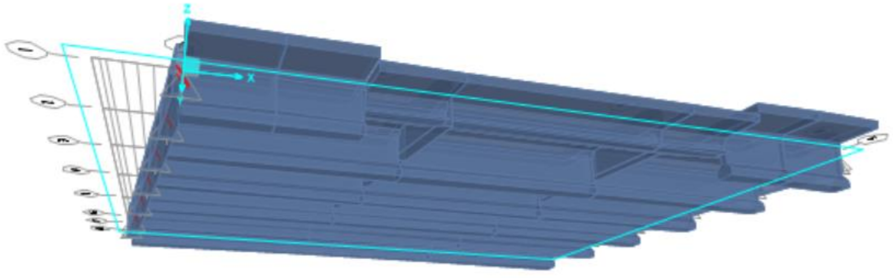
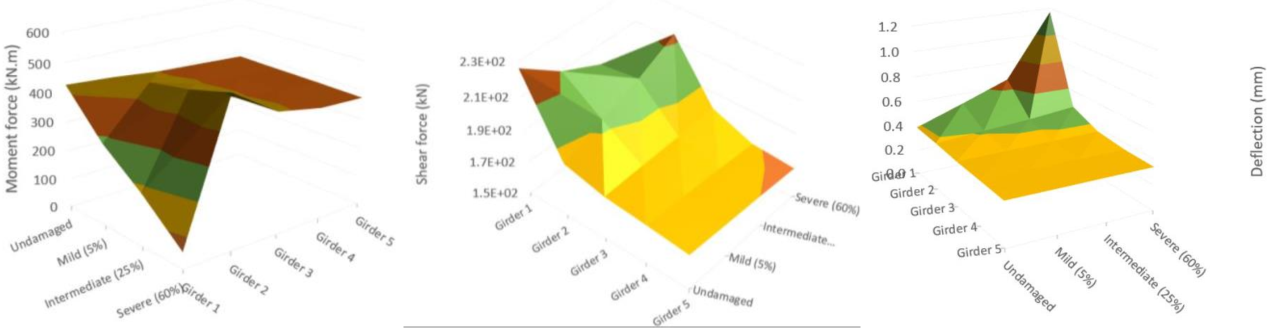


Fig. 9 - Finite Elements model simulating the damaged girder (Severe Damage)

After the damage, the moment, shear force and the deflection in the damaged girder and 4 adjacent girders will be calculated. This will show how the load will be redistributed after a damage to a certain level occurs while being excited under the same load conditions. The follwing tables show the redistribuion of the loads among five of the girders when damage is happening in the side girder. In order to see the load redsitribution ratios clearly, the ratio of reduction and increase are shown in the Figure 10.



(a) (b) (c)

Fig. 10 - Force redistribution and deflection on different girders for various severity of damage:
a) Bending Moment; b) Shear Force; c) Deflection

It can be seen from Figure 10 that when the damage happens in the side girder, the amount of force redistributed to it will be lower. The higher the damage severity, the lower the loads redistributed to it. This happens from losing stiffness of an element in a prallel spring problem. The softer an element, the lower the amount of force redirected on it. However, the deflection for the girder increases by the severity of the damage, which is the main reason of local collapse usually. It can also be seen that by increasing the severity of damage, the load in the adjacent girder to the damaged girder increasses highly. This increase would be less as of the damaged girder. The deflection would be also having the same behaviour. This shows that for this structure, the capcity of the elements that the redsitributed load on them increases (the adjaent elements in here), should be checked against their increased demand. Moreover, the deflection increase needs to be also taken into account for checking the safety of the bridge. Furthermore, the change of the first 6 freaquencies and the ratio of the change compared to the undamaged structure are shown in Table 4.

Table 4 - Frequency and frequency change ratios with different level of damage

Frequency (Hz)						
	Mode 1	**Mode 2**	**Mode 3**	**Mode 4**	**Mode 5**	**Mode 6**
Undamaged	12.4	12.81	14.74	22.5	31.98	32.78
Mild	12.3	12.7	14.8	22.7	28.95	32.07
Intermediate	12.0	12.6	14.6	22.7	25.07	32.06
Severe	11.5	12.57	14.55	19.3	22.8	32.1

Frequency Change Ratio						
	Mode 1	**Mode 2**	**Mode 3**	**Mode 4**	**Mode 5**	**Mode 6**
Mild	0.99	0.99	1.00	1.01	0.91	0.98
Intermediate	0.97	0.98	0.99	1.01	0.78	0.98
Severe	0.93	0.98	0.99	0.86	0.71	0.98

It can be seen from this analysis that the efficiency of using such approach to estimate the safety of continuing the operation of a bridge structure. It should be noted that althoguh the most affected elements in this simulation were the adjacent elements to the damage location, it mght not be the case for every structure. This needs to be checked by the updated FE model and accurate modelling and analysis based on the design maps and the real structure in place, in addition to observations on the condition of the damage and engineering judgment on it.

7. Proposed Framework for Safety Assessment of Damaged Bridge

The following step-by-step procedure is proposed for the comprehensive safety assessment of the bridge which has been damaged in an accident by over-height vehicle passing under the bridge. Figure 11 shows the flowchart of the proposed procedure for safety assessment of the bridge.

1. Conduct a series of Ambient Vibration Tests on Healthy Bridge (before accident), and perform modal analysis to identify the modal frequencies and mode shapes (This step is recommended for all important bridges in BC which have the highest level of risk of accident);
2. Develop the Finite Elements (FE) model of the Healthy Bridge. The FE model should be calibrated using the modal properties of the bridge obtained from Step 1;
3. Store the calibrated FE model of the Healthy Bridge and the modal properties in Data Library;
4. In case of accident, conduct full Ambient Vibration Tests on Damaged Bridge (bridge after accident), and perform modal analysis to identify the modal frequencies and mode shapes of the Damaged Bridge;
5. Simulate damage and modify the pre-calibrated FE model to develop the Damaged Bridge model. The FE model should be recalibrated using the modal properties of the Damaged Bridge obtained from Step 4 with only one variable representing the damage. Visual observation and engineering judgment of the damage should be used for damage location and intensity simulation in the model; In case of the unavailability of a pre-calibrated model, the FE model will be built and calibrated based on the AVT results after the damage and with two sets of variables, i.e. global variables and damage amount.

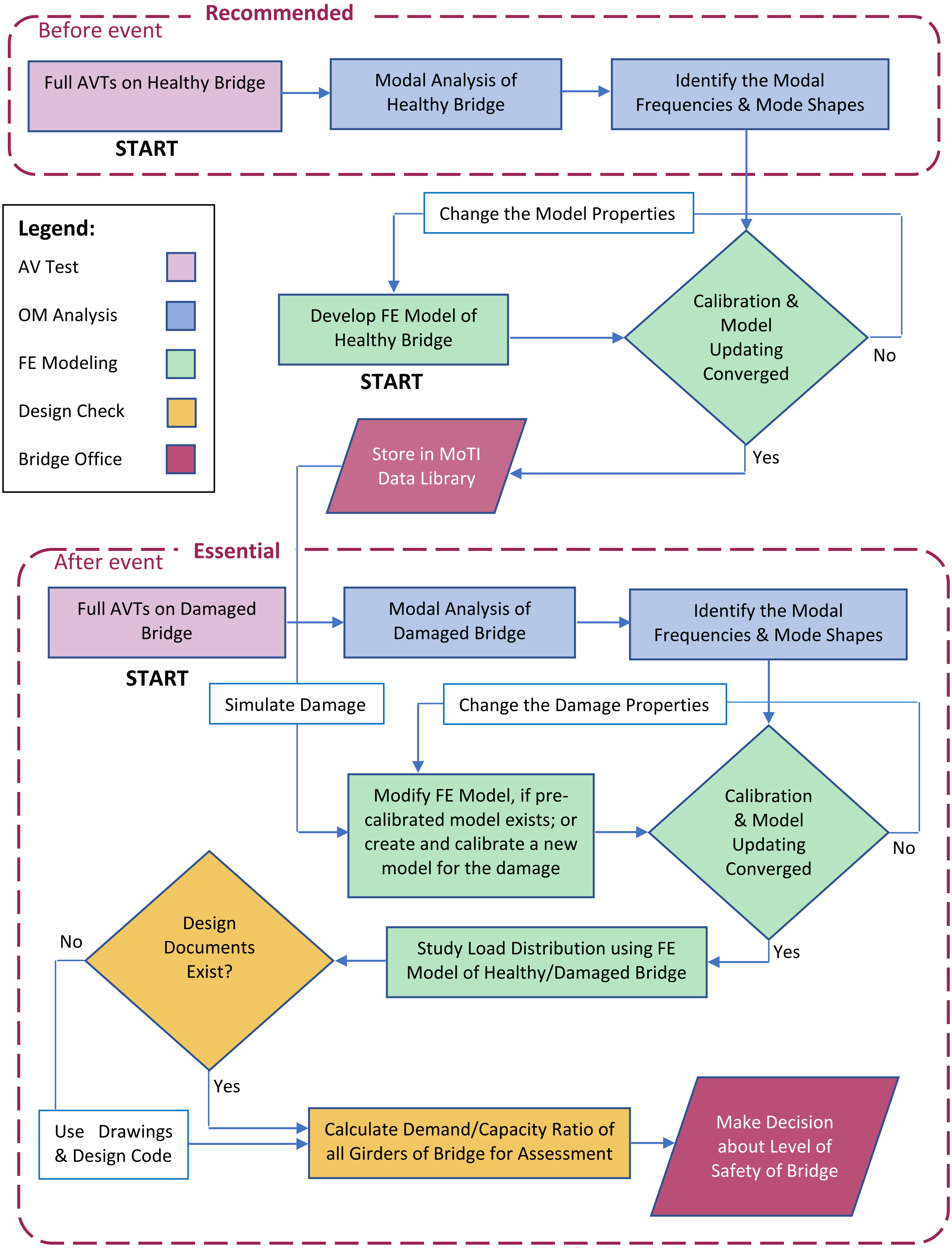


Fig.11 - Flowchart of the proposed procedure for comprehensive assessment of bridge

6. Study the load redistribution in the damaged bridge using FE model and compare to the healthy bridge to generate the redistribution ratios;
7. Use existing design documents of the bridge (or drawings and design codes in case of lack of the design documents) to calculate the demand/capacity ratio of the structural elements;
8. A licensed Professional Engineer shall make decision on the safety of the bridge according to the D/C ratio and the applicable Bridge Design Code requirements. The ultimate capacity or service capacity of the elements shall be chosen based on relevant service levels of the bridge.

8. Conclusions

A series of Ambient Vibration Tests was conducted on Huntington Bridge in order to determine the dynamic properties of the structure. The testing program consisted of five setups using wireless digital seismometer sensors placed on the deck in longitudinal direction of the bridge. Modal response analysis was performed using the computer program ARTeMIS to identify the natural frequencies and mode shapes of the deck. Two techniques; Frequency Domain Decomposition (FDD) and the Stochastic Subspace Identification (SSI) were used for modal identification. The results from modal analysis indicated that the frequencies of the deck are 11.4, 12.4, 14.8 and 21.3 Hz in vertical direction for the first 4 natural modes. A Finite Elements model was created using SAP2000 Software to simulate the dynamic behavior of the bridge. The boundary conditions of the deck were implemented in the model by rotational springs on the edges and updated based on the modal responses of the bridge obtained from AVTs. The identified natural frequencies and corresponding mode shapes of the deck were used to calibrate the analytical model. The calibrated model was used to estimate the effect of earthquake damage on the load distribution in the bridge. The redistributed loads were then computed and compared with the load capacity of the elements. A practical framework and step-by-step procedure were proposed for safety assessment of the damaged highway bridges. The calibrated Finite Element model was employed to examine the proposed framework. This framework can be used for rapid safety assessment of the bridge after earthquake to assure the safety of continuing the use of the bridge.

9. Acknowledgement

The ambient vibration tests, modal analysis and finite element analysis were conducted as a collaboration with the EERF personnel at the University of British Columbia and Ministry of Transportation and Infrastructures. This contribution is gratefully acknowledged.

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