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AMBIENT VIBRATION TEST AND SEISMIC PERFORMANCE ASSESSMENT OF PORTAGE CREEK BRIDGE

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Abstract

Constructed in 1983, the Portage Creek Bridge is a three span highway bridge located in Victoria, British Columbia (BC), Canada. This bridge is a part of a smart seismic monitoring program, British Columbia Smart Infrastructure Monitoring System (BCSIMS), funded by the Ministry of Transportation and Infrastructure (MoTI) BC, Canada. The BCSIMS is a real-time seismic monitoring program that assesses the seismic conditions of the selected bridges on lifeline highways in BC. The bridge was undergone a seismic retrofit in 2003. Most of the bridge was retrofitted by conventional materials and methods. An innovative retrofit technique-Fiber Reinforce Polymer Wraps (FRPs) was applied to strengthen the short column for shear without increasing the moment capacity. As part of the BCSIMS, an Ambient Vibration Test (AVT) was carried out on the bridge in September 2014, and the FEM of the bridge was created using Sap2000 [1]. The FEM of the bridge was updated using the AVT results, and the updated FEM was used to perform nonlinear static and dynamic analysis to assess the seismic performance of the bridge in accordance with the Canadian Highway Bridge Design Code, 2015.

Key words: ambient vibration test, modal analysis, finite element model updating, seismic evaluation

1. Introduction

The BCSIMS is comprehensive seismic monitoring program that integrates the Strong Motion Network (SMN) and the seismic Structural Health Monitoring (SHM) network in British Columbia (BC). The program was initiated in 2009 and involves fifteen structures (fourteen bridges and one tunnel) that are currently being monitored in real-time. One of the main intentions of the SHM network is to mitigate the seismic risk in bridges in BC by continuously assessing the seismic condition of the bridges, and it is done using the tools and techniques that have been developed over the last six years.

As part of the BCSIMS project, seismic evaluation was performed for Portage Creek Bridge, based on nonlinear time history analysis with selected ground motions. Structural conditions were assessed in accordance with the new Canadian Highway Bridge Design Code (CSA S06-14) which was released in 2014, by Canadian Standard Association (CSA). It incorporated new provisions for seismic evaluation of existing bridges.

2. Description and the Finite Element Model of the Bridge

The Portage Creek Bridge is a disaster-route bridge in Victoria, BC, Canada. It crosses the Interurban Road at McKenzie Avenue as shown in Figure 1. The bridge is 125 meters long and has three spans with concrete deck. The steel girders are supported using two reinforced concrete piers as can be seen in Figure 2. The deck has a roadway width of 16m (52ft) with two 1.78m (6’6’’) sidewalks and an aluminum railing on either side of the bridge. The bridge was designed in 1982, which is long before the introduction of current seismic design standards. As part of a seismic retrofit project, the seismic analysis of the bridge was performed by ISIS Canada, and seismic retrofit was carried out in 2003 in order for the bridge to meet the seismic design requirement in that time. An innovative retrofit technique-Fiber Reinforce Polymer Wraps (FRPs) was applied to strengthen the short column for shear [3]. With the structural aging and introduction of new seismic design provisions in 2015, the bridge is in need of a re-assessment of seismic performance in accordance with the new Canadian Highway Bridge Design Code, 2015.

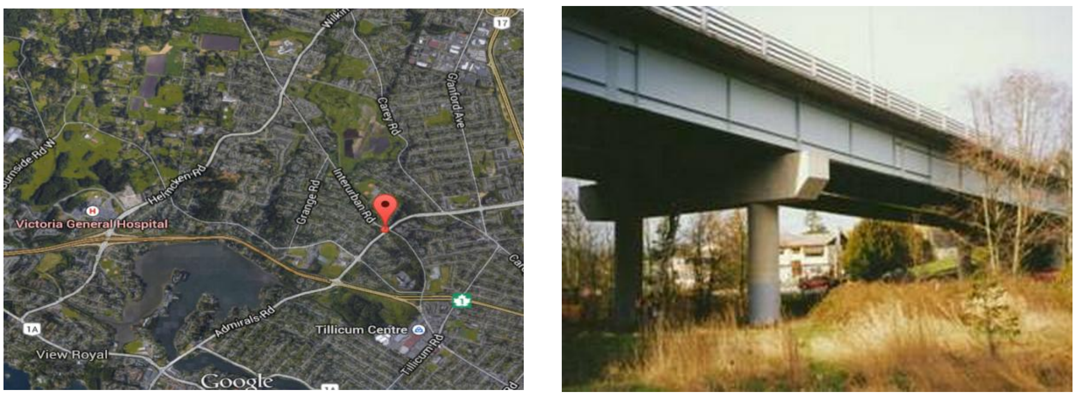


Fig.1– Location and view of Portage Creek Bridge

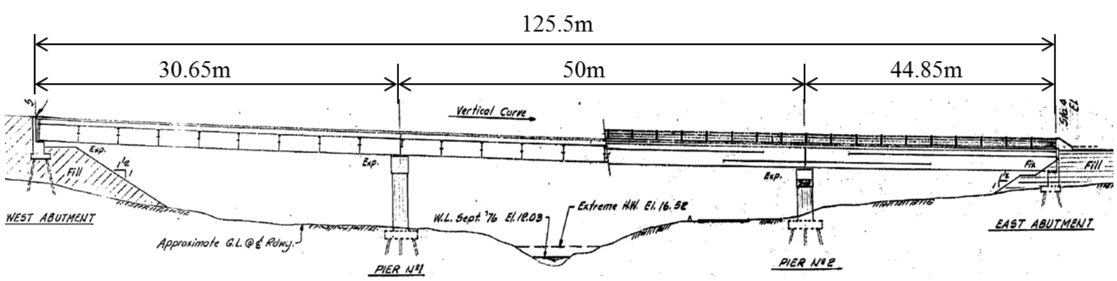


Fig. 2 – Elevation of Portage Creek Bridge

The FE model of the bridge was built in Sap2000 as seen in Figure 4. The Geometry and the material properties of the structure were obtained from the original structural drawings and the retrofit project in 2003. Structural members were modeled in different ways in Sap2000. Shell area elements are used for concrete slab. The cross section of the concrete slab is uniform within each span but differs between spans. Three steel girders and four small steel stringers support concrete slabs. Steel beams connect the girders and stringers. The girder section is changing over the span while stringer and beam section remain unchanged. Girder, stringer, and beam are all made of H-shape steel and are modeled as frame element. Concrete bents and piles that support the superstructure were simply modeled as frame element. There are sliding bearings between concrete bents and steel girders to dissipate energy during earthquake. The bearings were modeled as a link element, which is free to slide in translational directions and fixed in other DOFs. Due to lack of detailed material properties, the stiffness of the link element is estimated by equations proposed by Akogul [4]. Link element is used to model the soil conditions: the soil-spring model is referred as Winkler model [5]. Stiffness of these soil springs are estimated following the instruction of Das [6].

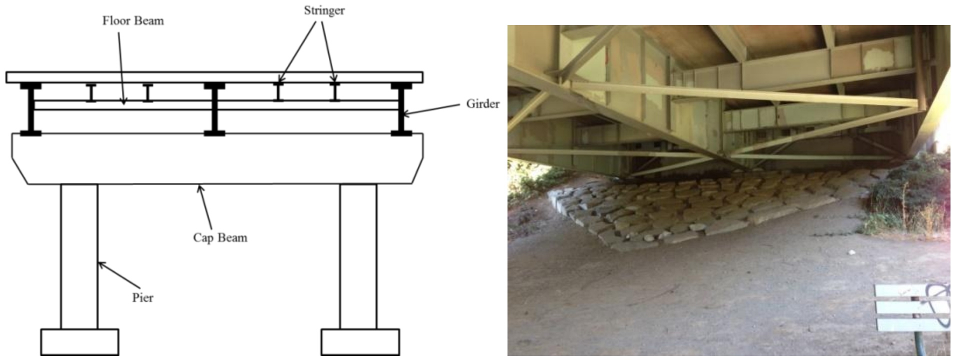


Fig. 3 – Cross-section of Portage Creek Bridge and location of bracing

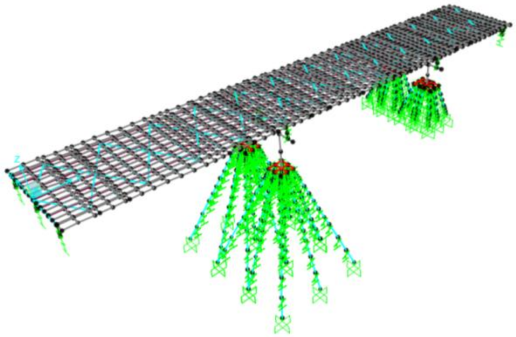


Fig. 4 – Finite element model of Portage Creek Bridge

3. Ambient Vibration Test and Modal Analysis

In order to make the seismic evaluation more reliable, Ambient Vibration Testing (AVT), which is a non-destructive vibration testing technique aiming to identify the dynamic characteristics of structures, was carried out by a research group in Earthquake Engineering Research Facility (EERF) at UBC. The finite element model used for seismic evaluation was calibrated based on the dynamic characteristics obtained from AVT. Structural vibration sensors, TROMINO sensors as seen in Figure 5, were used to carry out the AVT of the Portage Creek Bridge.

Fig. 1 – TROMINO sensor with radio

All of the TROMINO sensors are placed on the sidewalk of the bridge, and one radio amplifier is connected to each sensor as shown in Figure 5. In order to capture the higher modes with good accuracy, 32 testing points are selected at different locations of the bridge. The test was divided into five setups, and eight sensors were used for each setup (Figure 6). One sensor located at the mid-span of the bridge was used as reference sensor for all setups. The length of the recording for each setup is 30 minutes with a sampling frequency of 128 Hz. Sensor locations for five setups are highlighted in Figure 6. The AVT test was started at 14:25 pm and was finished at 17:34 pm. The recorded data from all setups are synchronized using the Grilla software [7]. Only the high gain velocity records are used in the modal analysis because of the limitations of Grilla software in the data synchronization.

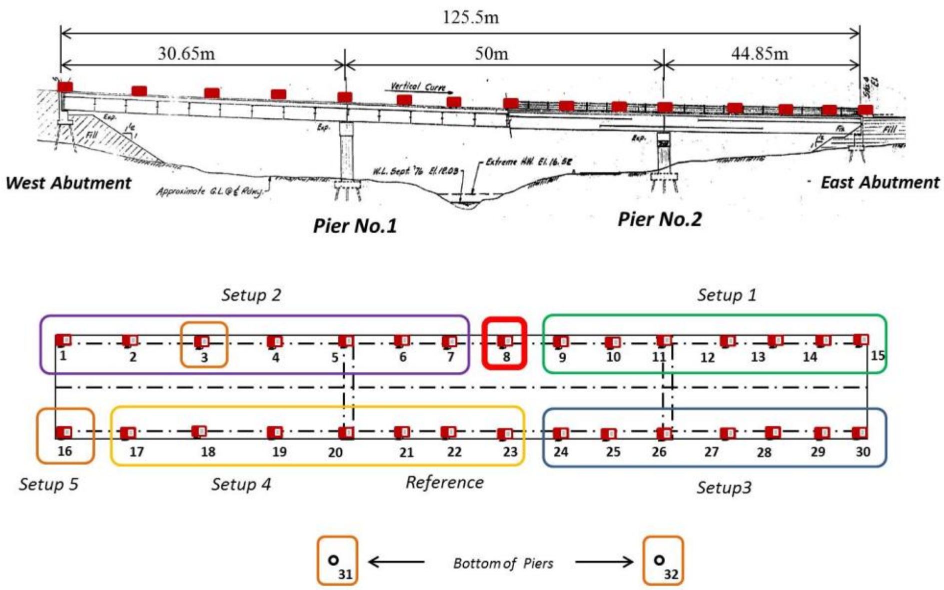


Fig. 2 – Locations for all test setups

A commercial software ARTeMIS [8] was used for data processing, modal identification and the visualization of mode shapes. The raw data from each setup is first baseline corrected and low-pass filtered with a cutoff frequency of 20Hz. Enhanced Frequency Domain Decomposition (EFDD) techniques is use to extract the modal properties of the bridge. Modal frequencies, mode shapes and damping were estimated though peak-picking spectral densities, and the extracted modes are listed in Table 3.

4. Finite element model updating

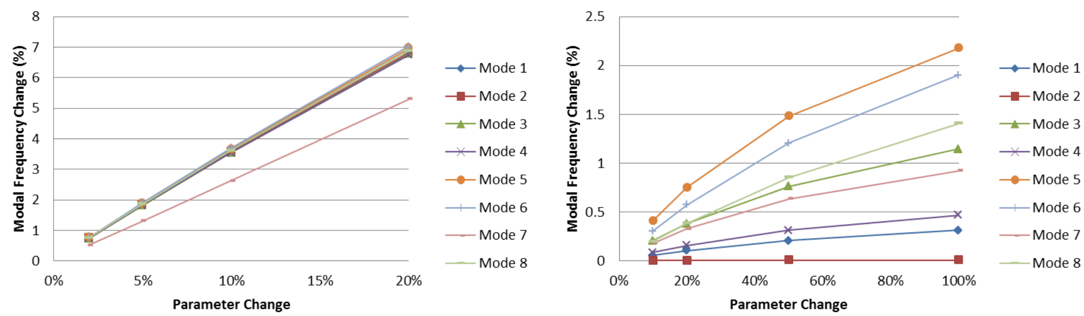
Due to the uncertainties in the material properties, bearings, soil condition, and boundary conditions, the FE model of the bridge does not represent the current condition of the bridge; therefore, the FE model of the bridge is updated (calibrated) using the AVT.

The key to success in model updating is the choice of parameters. The parameters should be selected where uncertainties are likely to arise. Table 4 summarizes the selected parameters for model updating.

Table 2 – Parameters chosen for FE model updating of the bridge

Element	Deck		Girder		Column		Foundation	Expansion Bearing
type	E	ρ	E	ρ	E	ρ	k	k

Updating the FE model manually is basically a trial and error approach, but it is very time consuming. The process is expedited by running a sensitivity analysis on the selected parameters of the FEM model. 8 sets of sensitivity analysis were performed for all the parameters listed in Table1. Figure 8 shows typical results from sensitivity analysis. It is found that the sensitivity analysis results show different characteristics for material properties and link properties, as shown in Figure 8. For material properties, all the modes have similar sensitivities to certain material parameter, so it do not help a lot when I want to adjust the frequencies of some modes and leave those of other modes unchanged. For instance, if the 1st mode from FE model matches the test results perfectly while the 3rd mode does not, it is impossible to achieve a good match for both modes through adjusting the material parameters. However, different modes show a variety of sensitivities towards link properties, which makes it possible to calibrate the model when encountering the situation mentioned above.



a) b)

Fig.8 – Typical results from sensitivity analysis of **a)** material property and **b)** link property

Based on the findings from the sensitivity analysis, the FE model of the bridge was updated manually. For example, the sensitivity analysis shows the 2nd mode is most sensitive to the bearing stiffness in translational directions, so the frequency of 2nd mode was calibrated by adjusting bearing stiffness manually. The final results are tabulated in Table 3. The comparisons of mode shapes between numerical and experimental results are shown in Figure 9.

Table 3 – Modal properties of the Portage Bridge before and after FE model calibration.

Mode No.	FEM (Hz)	ARTeMIS (Hz)	Damping (%)	Updated FEM (Hz)	Diff (%)	Mode Description
1	2.416	2.507	3.19	2.442	2.59%	1st Vertical Direction
2	2.664	2.725	3.189	2.763	1.39%	1st Torsional
3	3.275	3.165	3.15	3.258	2.94%	2nd Vertical Direction
4	3.308	3.375	3.154	3.356	0.56%	2nd Torsional
5	7.031	6.96	1.175	7.09	1.87%	3rd Torsional
6	7.897	7.56	2.404	7.969	5.41%	3rd Vertical Direction
7	8.377	8.375	2.862	8.499	1.48%	4th Vertical Direction
8	11.235	11.14	0.881	11.368	2.05%	5th Vertical Direction

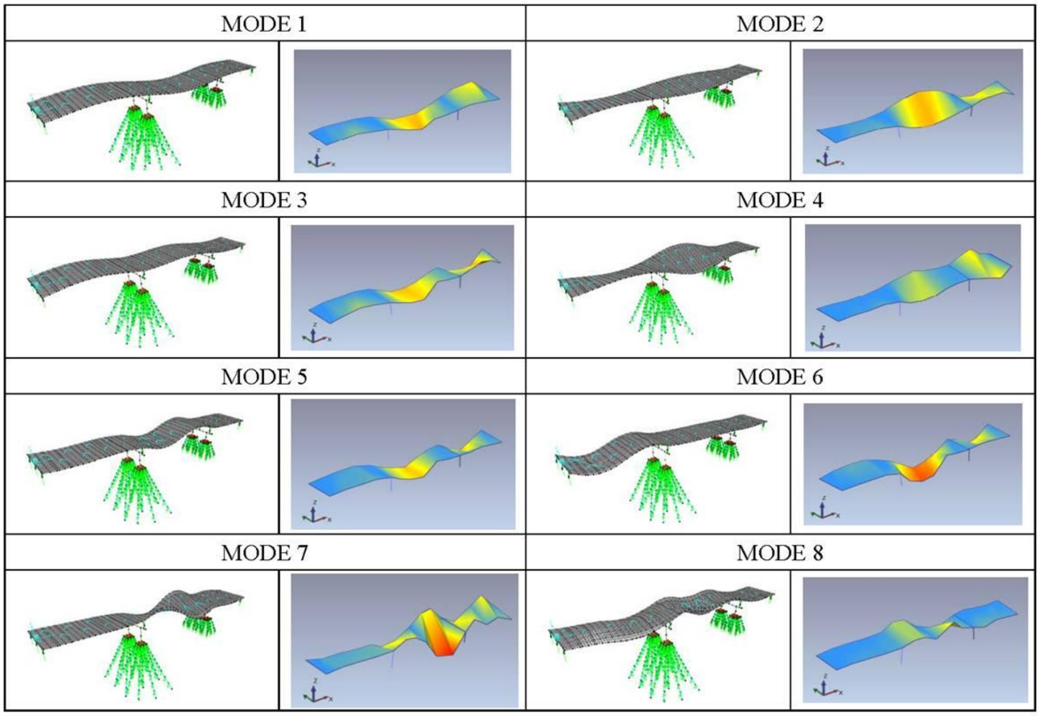


Fig.9 – Comparison of mode shapes between numerical and experimental results

5. Seismic Performance Assessment

5.1 Performance criteria

According to Table 4.12 of CSA S06-14, performance-based evaluation is required for Portage Creek Bridges given its importance and seismic performance categories. Performance level shall be determined first in performance-based design approach. There are four performance levels specified in Table 4.16 of CSA S06-14, which are Immediate, Limited, Service Disruption and Life Safety. Performance criteria about service requirements and expected damages are described in the code for each performance level. Performance levels shall be satisfied under earthquake for different return periods. For Portage Creek Bridge, the first performance levels shall be satisfied for return periods of 475 years and 975 years, which means an immediate return to occupancy is expected during small to moderate earthquake. During severe earthquake with return period of 2475 years, the second performance level is required which means limited damage shall occur but the damage is repairable without requiring bridge closure.

5.1.1 Concrete Structures

Performance criteria for concrete structure are strain limits for concrete and reinforcing steel. The strain varies over the cross-section for flexural members. Assuming that plain section remains plain after bending, strain at a distance of y from neutral axis can be expressed as:

$$\varepsilon = \frac{M}{EI} y \tag{1}$$

where E is the elastic modulus of the material; I is the second moment of inertia; M is the bending moment about the neutral axial. The strain-moment relationship can also be obtained from section analysis. Then the relationship between maximum strain and moment at different axial force (P) values can be determined, for pier concrete and pier rebar.

Therefore, the strain requirements in the performance criteria can be converted to moment requirement based on the moment-strain curve. Taking the first performance level for example, the performance criteria require the maximum concrete compressive strain shall not exceed 0.004 and reinforcing steel strain shall not yield. For piers, the critical bending moments at concrete compressive strain of 0.004 and rebar yield strain are identified at different axial force values. The interaction between the critical bending moments and axial forces can be viewed in Figure 12 and 13 for Pier No.1 & 2 respectively. These interaction curves can be regarded as the acceptance criteria for the first performance level and will be compared with the resulted element moment from seismic analysis. For cap beams, axial loads were not considered in the analysis. The critical moments for each performance level are summarized in Table 4.

Table 4 – Critical moments for cap beams (kN•m)

	Performance Level 1	**Performance Level 2**	
	Rebar Yield	Concrete strain 0.004	Rebar Strain 0.015
Positive	11050	12610	14240
Negative	33100	35800	37800

Shear capacity of concrete structures can be calculated according to the equations specified in CSA S06-14. The calculated shear capacities of cap beam and pier are summarized in Table 5.

Table 5 – Nominal shear capacity of primary structural elements

	Pier No.1	**Pier No.2**	**Cap Beam**
Shear Capacity (kN)	5303	8010	4935

5.1.2 Steel Structures

Steel structure in Portage Creek Bridge, which includes Girders, stringers and floor beams, are all secondary structures. The buckling of steel structure was checked by Euler's critical load [10]:

$$F = \frac{\pi^2 EI}{(KL)^2} \tag{2}$$

where F is the expected compressive force on buckling; E is modulus of elasticity; I is area moment of inertia of the cross section; L is the unsupported length of the element and K is the effective length factor. Girders and stringers are all casted together with the concrete decking, which means their unsupported lengths are zero and the buckling fore is infinite large. Floor beams and braces were regarded as fixed-end elements so the K factor was 0.5. Element properties and buckling forces of floor beams and braces are summarized in Table 6.

Table 6 – Buckling force of brace and floor beam

	E (Mpa)	$I\ (mm^4)$	$L\ (mm)$	$F\ (kN)$
Brace	200000	9105150	9485	798
Floor Beam	200000	142800000	2692	155427

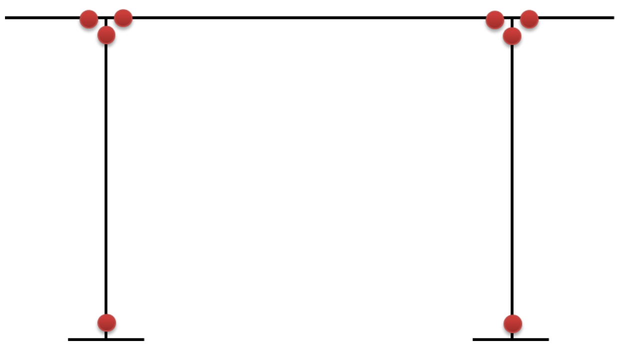
Primary connections of the bridge are the connections between pier and cap beam as well as the connections between pier and foundation footings. Connections were checked for capacities of sections around them. Performance of elastomeric bearings, structural displacement and foundation movement was obtained from seismic analysis and evaluated in terms of the criteria in CSA S06-14. There is no restrainer in this bridge.

5.1.3 Bearing, Displacement and Foundation

CSA S06-14 allows the failure of bearings, so the responses of bearings are not presented in this paper. Residual displacements and foundation movements shall be limited to a small level according to CSA S06-14.

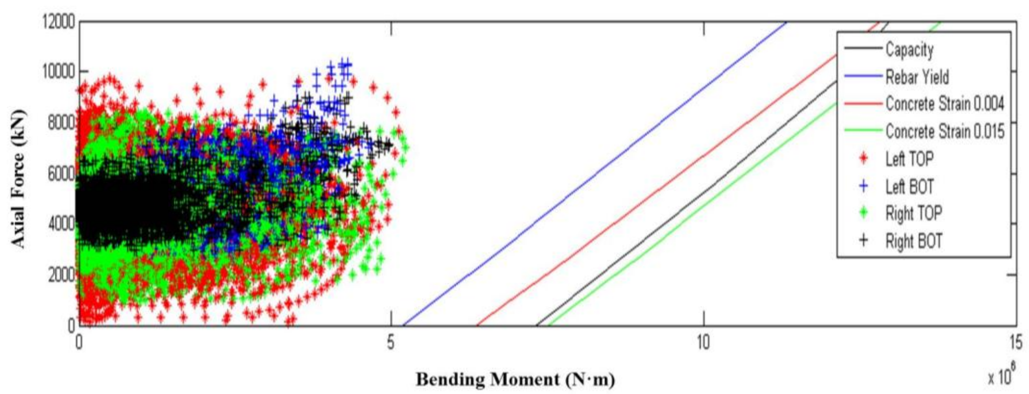
5.2 Seismic Analysis

Nonlinear analysis model was established in SAP2000 with concentrated plasticity. Plastic hinges were assigned at the end of piers and cap beams, as shown in Figure 10. Properties of plastic hinges were determined through section analysis.

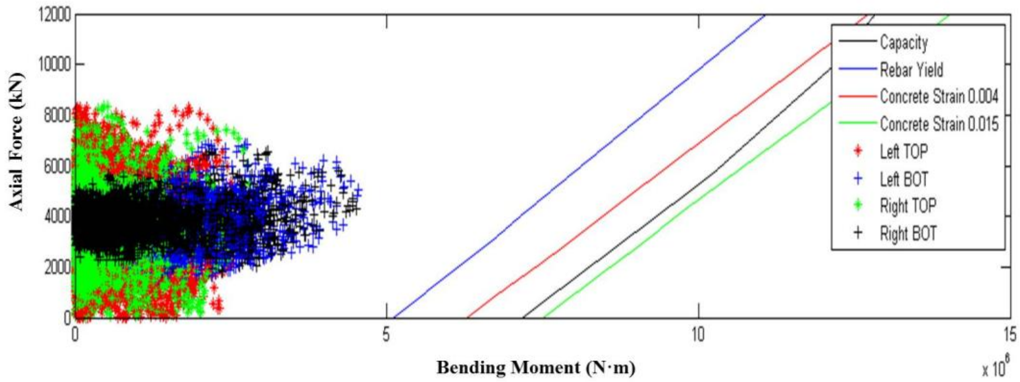


Fg.10 – Locations of plastic hinges

Nonlinear time-history analysis was then performed to predict the seismic performance of the bridge. A total of 11 sets of analysis were carried out, with different ground motion inputs for each sets. Flexural responses for piers at each time step are all plotted in Figure 11 & 12 with the member capacities and performance criteria described in Section 5.1. For shear response of piers and dynamic response of cap beams, only maximum responses at critical sections are identified and summarized in Table 7 & 8.



Fg.11 – Summary of flexural response for Pier No.1 (NLTHA)



Fg.12 – Summary of flexural response for Pier No.2 (NLTHA)

Table 7 – Summary of shear response for piers (NLTHA)

		S22	S33	Max.S	D/C
No.1 Right	TOP	696.21629	1659.2841	1659.2841	**0.312895**
	BOT	959.7736	1361.6674	1361.6674	**0.256773**
No.1 Left	TOP	854.24516	1726.8703	1726.8703	**0.32564**
	BOT	1086.0015	1571.6341	1571.6341	**0.296367**
No.2 Right	TOP	1782.23752	1877.5769	1877.5769	**0.354059**
	BOT	2067.58958	1545.7153	2067.5896	**0.389891**
No.2 Left	TOP	1855.32783	1852.2071	1855.3278	**0.349864**
	BOT	2141.70321	1877.5769	2141.7032	**0.403866**

Table 8 – Summary of cap beam response (NLTHA)

		Mmax	D/C	Mmin	D/C	S22	D/C
No.1 Right	**Ext**	-88405.5	-0.006208	-10802156	0.285771	4644.691	0.941173
	Int	5449431	0.3826848	-18014237	0.476567	4947.628	1.002559
No.1 Left	**Ext**	428430	0.0300864	-12208318	0.322971	5212.149	1.05616
	Int	1929263.6	0.135482	-13552258	0.358525	3738.516	0.757551
No.2 Right	**Ext**	2319922.1	0.1629159	-10399390	0.275116	4538.77	0.91971
	Int	5481027.9	0.3849036	-13048112	0.345188	4135.602	0.838015
No.2 Left	**Ext**	1552709.6	0.1090386	-10785238	0.285324	4519.619	0.91583
	Int	2036131	0.1429867	-11757735	0.311051	3556.614	0.720692
MAX		**5481027.9**	**0.3849036**	**-10399390**	**0.476567**	**5212.149**	**1.05616**

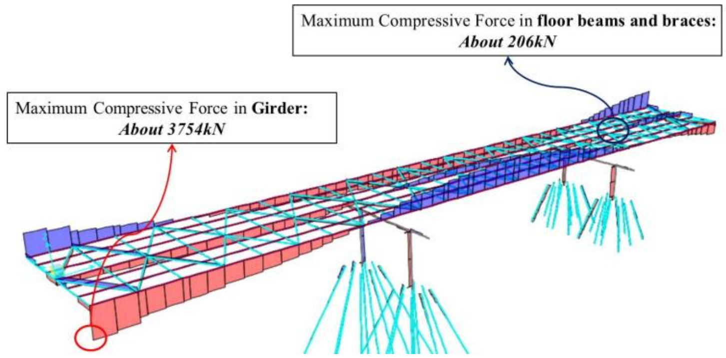


Fig.13 – Axial force diagram under Tohoku earthquake

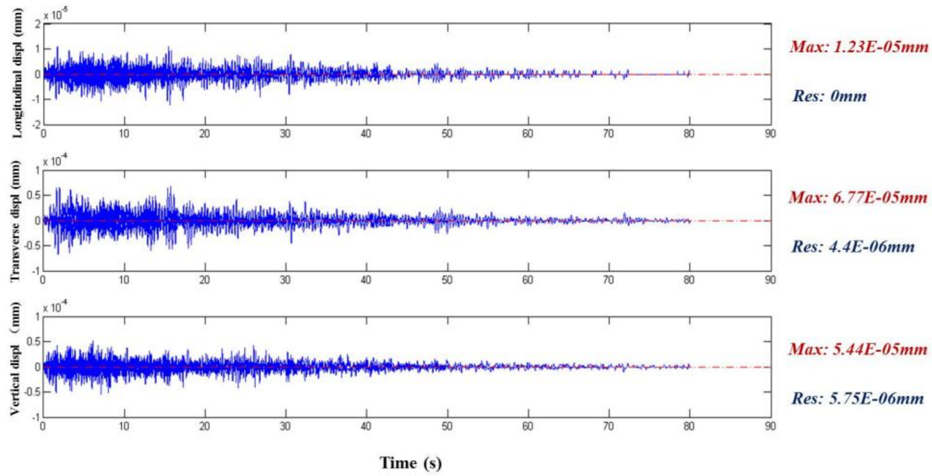


Fig.14 – Displacement response at west-abutment

5.3. Discussion of results

Performance of Portage Creek Bridge can be evaluated in terms of the criteria described in Section 5.1. The evaluation for each structural aspect is presented below:

1) Concrete structures and connections: Concrete compressive strains do not exceed 0.004 and reinforcing steels do not yield at all the critical sections of substructure, as shown in Figure 11 & 12 and Table 7 & 8. However, the cap beam at Pier No.1 has a risk of failing in shear at its connections with piers.

2) Steel Structures: Steel structures are all assigned in superstructure so they are secondary members. CSA S06-14 allows the buckling of secondary steel members but the structural instability is not permitted. Since the girders and stringers are casted together with the concrete decking, their stability can be ensured. For the floor beams and braces, the analysis results shows that both of them will not experience large axial forces under major earthquakes, so the instability issues are not possible to occur. Axial force diagram obtained from seismic analysis under Tohoku earthquake records is shown in Figure 13 as example. It shows that the maximum axial force in braces and floor beams is only about 200kN which is much smaller than the buckling forces calculated from Euler's equation.

4) Displacements and foundation movements: Analysis result shows that there are no significant residual displacement or foundation movements during severe earthquake. The displacement responses at west-abutment of the bridge are shown in Figure 14.

6. Conclusion

The results of the study show that during severe earthquake: 1) there will be some inelastic behavior for the primary members (piers, cap beams and girders) of the bridge but the moment capacities of these members meet the demands; 2) the shear capacity of cap beam at Pier No.1 is not adequate. Since shear failure is brittle failure which is not convenient to be repaired in place, the seismic performance of cap beam is not acceptable according to CSA S06-14; 3) there are no permanent offsets and residual displacements for both the superstructure and foundation so the displacement-related criteria are satisfied.

Given the fact that the cap beam at Pier No.1 has a high risk of failure in shear which is fatal in severe earthquake and cannot be repaired in place after earthquake, Portage Creek Bridge does not meet the seismic performance criteria specified in Canadian Highway Bridge Design Code. Seismic retrofit is necessary to ensure the safety of this bridge in potential major earthquake.

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