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EVALUATION OF PERFORMANCE-BASED SEISMIC RETROFIT OF BRIDGE PIERS WITH PSEUDO-DYNAMIC TESTS

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ABSTRACT

New uniform hazard spectra, at 2% in 50 years probability level, have been developed by the Geological Survey of Canada (Adams and Halchuk, 2003) for the 2005 edition of the National Building Code of Canada (NBCC 2005). This change from 10% to 2% in 50 years probability level has a significant impact on moderate seismic activity regions like Eastern Canada. This impact is studied in a bridge pier seismic retrofit project which is currently underway at the Earthquake Engineering and Structural Dynamics Research Centre (CRGP) at the Université de Sherbrooke. The main objectives of the project presented herein are to optimize a retrofitting methodology of bridge columns with carbon fiber reinforced polymers (CFRPs) and to evaluate the earthquake performance of a bridge bent – before and after retrofit – by means of pseudo-dynamic testing (PSD) with substructuring. This method, which represents the state-of-the-art in earthquake testing, was used to develop a performance-oriented test protocol considering input motions corresponding to various limit-states of a bridge.The prototype selected for the tests is a typical regular highway bridge with two three-column bents located in Trois-Rivières in the province of Quebec. A large-scale (1:3) model of the three-column bridge bent was subjected to a total of 5 simulated earthquake loadings corresponding to 3 increasing levels of intensity. The bridge bent specimen was subjected to the first level of earthquake intensity. Then, two of the three columns were retrofitted with CFRPs according to the optimized retrofitting scheme before submitting the bridge bent specimen to tests 2 to 5. The retrofitting methodology is based on performance criteria i.e., the retrofitted structure must meet prescribed ductility and drift requirements corresponding to given seismic events having respectively low, medium and high probability of exceedance. Also, a new confinement model for the behaviour of circular concrete columns confined with transverse steel and CFRPs (Eid and Paultre, 2006) is included in the retrofitting methodology. Test results show that the design procedure is conservative and the data acquired during this series of test can be referred to when establishing quantitative criteria for performance-based retrofit methods. Results of on-site dynamic test under ambient vibration performed on the actual bridge were used to update a 3D finite element model of the bridge. This model, developed with the program Ruaumoko and the sectional analysis program WMNPhi, proved to be effective in predicting the non-linear response of the bridge bent.

Introduction

New uniform hazard spectra, at 2% in 50 years probability level, have been developed by the Geological Survey of Canada (Adams and Halchuk, 2003) for the 2005 edition of the National Building Code of

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Canada (NBCC 2005). This change from 10% to 2% in 50 years probability level has a significant impact on moderate seismic activity regions like Eastern Canada. This impact was studied in a bridge pier seismic retrofit project, in which the objectives were to optimize a retrofitting methodology of R/C bridge columns with carbon fiber reinforced polymers (CFRPs), based on performance objectives, and to evaluate its earthquake performance by means of pseudo-dynamic (PSD) testing with substructuring. The bridge selected for PSD testing is shown in Figure 3 and is located in a moderate seismic activity region of the province of Quebec. For this particular region, the peak ground acceleration (PGA) was 0.12g for a probability of exceedence of 10% in 50 years (return period of 475 years), as prescribed in the National Building Code of Canada (NBCC 1995) and in the actual Canadian Highway Bridge Design Code (CHBDC 2000). This value has been modified to 0.18g due to the adoption of a new hazard model by the Geological Survey of Canada. Moreover, for a probability of exceedance of 2% in 50 years (return period of 2500 years), on a firm soil site, the PGA prescribed for the bridge location in the new Canadian code (NBCC 2005) is 0.40g (Adams and Halchuk, 2003). The seismic hazard to be considered for this bridge has therefore changed significantly since it was designed and, based on this fact alone, a retrofit of the bridge would be required. However, the seismic evaluation of the structure showed that the bridge bent has sufficient ductility capacities to resist this increase in seismic demand. Therefore, in order to demonstrate the efficiency of the retrofitting methodology, the bridge bent specimen was also submitted to the hazard of a higher seismic activity region of the province of Quebec.

Evaluation of Capacity and Demand

An evaluation of the seismic vulnerability of the bridge was performed with the N2 method (Fajfar, 1999). This method combines the capacity spectrum method – which graphically compares the capacity of a structure with the demands of specified earthquake ground motions – with the use of inelastic demand spectra. The demand consists in uniform hazard spectrum having return periods of 475 and 2500 years for a region of moderate seismic activity in eastern Canada and 2500 years for a high seismic activity region. The performance matrix corresponding to this experimental program is shown in Figure 1.

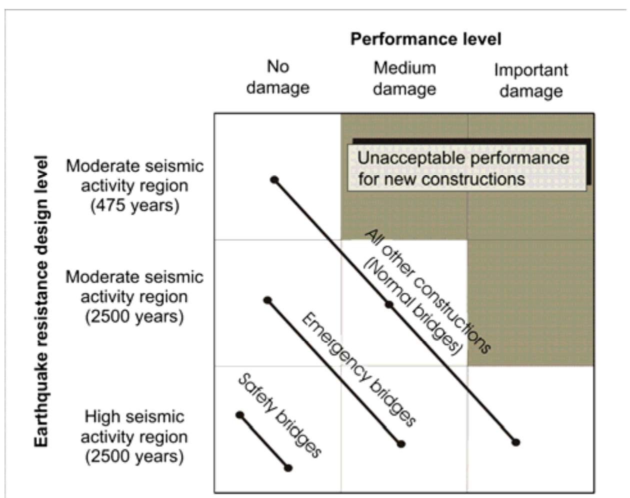

Figure 1. Performance matrix.

The ductility capacity in displacement μ_Δ of the bridge bent is defined as $\mu_\Delta = \Delta_u / \Delta_y$ where Δ_u is the ultimate top lateral displacement and Δ_y is the yield top lateral displacement of the bridge bent. The monotonic ductility capacity in displacement $\mu_{\Delta m}$ was obtained from a pushover analysis (monotonic loading) performed on an updated 3D numerical model of the whole bridge developed with the non linear analysis Ruaumoko program (1998). The monotonic ductility capacity in displacement $\mu_{\Delta m}$ was reduced with due considerations to cumulative damage in order to obtain the cyclic ductility capacity $\mu_{\Delta c}$ using the following expression proposed by Park et al., 1984:

$$\frac{\mu_{\Delta c}}{\mu_{\Delta m}} = \frac{1}{1+\beta\gamma^2\mu_{\Delta c}} \tag{1}$$

where β is a strength degradation parameter and is taken equal to 0.15 and γ is a parameter related to the ratio between the dissipated hysteretic energy and the maximum displacement (taken equal to 1). The computed cyclic ductility capacity $\mu_{\Delta c}$ for the bridge was found to be equal to 1.52. According to the N2 method, the seismic demand was obtained from a comparison between the uniform hazard spectrum considered and the idealized force-displacement response of the structure. This response was converted into an elastic – perfectly plastic curve of an equivalent SDOF system. As can be seen in Figure 2, in the high seismic activity region and for a return period of 2500 years, the demand in terms of displacement ductility $\mu_{\Delta d}$ of the bridge bent exceeds its capacity and is equal to 1.92. In the next section, retrofit of the bridge bent columns is designed in order to meet this demand.

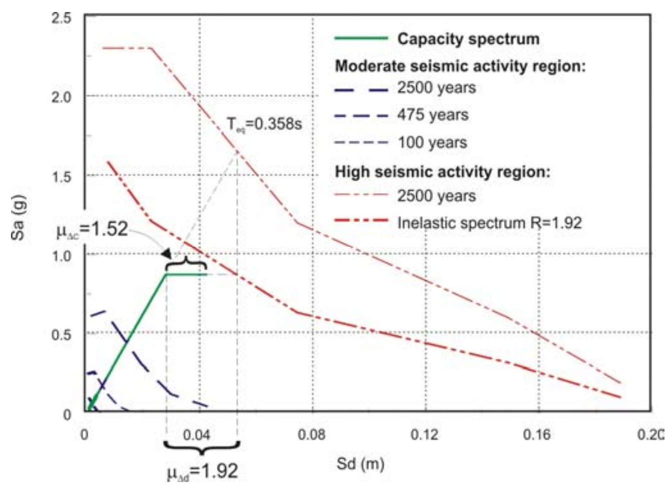

Figure 2. Capacity and demand spectrum.

Retrofit Design Methodology

In order to increase ductility capacities in displacement of R/C columns, the design of the retrofit consists of the following steps (Priestley et al., 1996): (i) calculate the plastic hinge length, l_p; (ii) determine the required curvature ductility, μ_φ; (iii) calculate the corresponding required maximum compression strain ε_{demand}; and, (iv) determine the ratio of confinement required. The curvature ductility demand μ_φ is calculated with the following geometric relationship (Park and Paulay, 1975):

$$\mu_\varphi = \frac{\varphi_u}{\varphi_y} = 1 + \frac{\mu_\Delta - 1}{3\frac{l_p}{l}\left(1-\frac{l_p}{2l}\right)} \tag{2}$$

where φ_u and φ_y are the ultimate and yield curvature of the concrete section, l is the length between the end of the member and the point of contraflexure and l_p is the equivalent plastic hinge length given by Priestley et al., 1996 as:

$$l_p = g + 0.044 d_b f_y \tag{3}$$

where g is the gap between the jacket and the supporting member, d_b and f_y are the diameter and the yield strength of the longitudinal bars, respectively. The maximum concrete compression strain required can be obtained from the following:

$$\varepsilon_{demand} = \varphi_{demand} c \tag{4}$$

where c is the neutral axis calculated with the sectional analysis program WMNPhi (2000). This software is used to predict the moment-curvature response using several stress-strain models for the reinforcement and the confined and unconfined concrete. The number of layers of CFRP required is calculated with a material-dependent relationship between ultimate compression strain and volumetric ratio of jacket confinement. The relationships used in this project are derived from the Eid and Paultre confinement model (Eid and Paultre, 2006). This new model presents two important advantages : (i) it considers the two actions of confinement on the concrete section, i.e., the action due to the CFRP and the action due to steel ties and (ii) the definition of the confinement action due to CFRP was expressly derived for composite material (and not from equivalent steel). In this new model, the ultimate strain of the section confined with CFRP, $\varepsilon_{provided}$, is calculated with the following expression derived by Lam and Teng, 2004:

$$\varepsilon_{provided} = \varepsilon_{c0}\left[1,75 + 5,53\left(k_e \frac{A_{sh} f_{hy}}{s D_c f_{c0}} + \frac{2tE_f \varepsilon_{fu}}{D f_{c0}}\right)\left(\frac{\varepsilon_{fu}}{\varepsilon_{c0}}\right)^{0.45}\right] \tag{5}$$

where f_{c0} and ε_{c0} are the unconfined concrete strength and its corresponding strain, k_e is a coefficient introduced by Sheikh and Uzumeri (1982) and Mander et al. (1988) which reflects the effectiveness of the lateral steel in confining the concrete, A_{sh} is the total cross-section area of the transverse reinforcement, f_{hy} is the yield stress of the transverse reinforcement steel, s is the steel tie spacing, D_c is the concrete core diameter, D is the full column diameter and t, E_f and ε_{fu} are the thickness, the elastic modulus and the ultimate tensile strain of the CFRP. Considering the material chosen for the confinement, the retrofitted columns have a cyclic displacement ductility capacity of $\mu_{\Delta c}$ = 2.80 which exceeds the displacement ductility demand of $\mu_{\Delta d}$ = 1.92.

Pseudo-Dynamic Test with Substructuring

The experimental evaluation of the retrofitting scheme was carried out using the pseudo-dynamic technique with substructuring. This method, which represents the state-of-the-art in earthquake testing, was used to develop a performance-oriented test protocol considering input motions corresponding to various limit-states of the bridge. A pseudo-dynamic test is a hybrid test where the restoring forces of a structure are determined experimentally, while the time-dependent forces, i.e. damping and inertia, are simulated numerically. The test can thus be performed in a quasi-static manner, which is much simpler than a real-time test. As the tests are run slowly, inspection is done troughout the test. In the substructure method, only a selected portion of a structure is tested, while the rest is modelled numerically (Pinto et al., 1996).

Substructuring is implemented by two different processes. The PSD controller process is responsible for simulating the dynamic effects of the 1:3 scaled model of the three-column bridge bent and for controlling the testing machine, while the substructuring process simulates the linear part of the structure – the deck in this case – by a finite element model (FEM) (Figure 3). The two processes have to exchange information on the common degrees of freedom: the PSD controller process sends the displacement values to the substructuring process and receives the force values from the finite-element process. The time-integration scheme adopted in this project is based on the α-method which is an unconditionally stable implicit algorithm (Hilber et al., 1977).

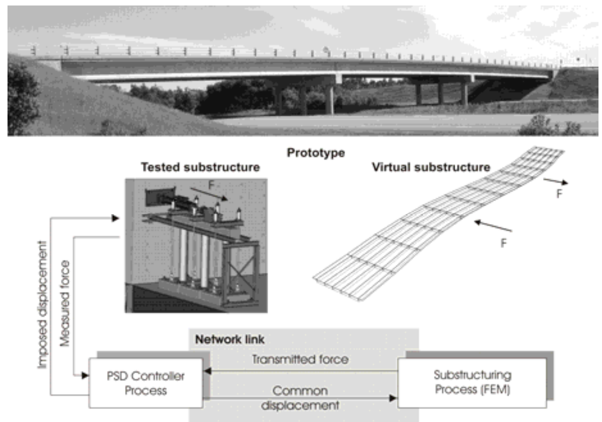

Figure 3. Pseudo-dynamic testing with substructuring.

Experimental Program

A scale factor of 3 was chosen for the model to accommodate the facilities of the Sherbrooke laboratory. Similitude relationships were used between the actual bridge and the model. The geometric characteristics of the bent model are presented in Figure 4. The bridge bent specimen is fully instrumented with strain gauges to measure the deformations in the longitudinal and transverse reinforcement in the columns. Displacement transducers measure top lateral displacement, joints displacement and curvature at top and bottom of the columns. The bridge bent specimen was subjected to increasing simulated earthquake loading as described in Table 1. The lateral seismic load is applied to the bridge bent by a double-acting dynamic-rated servo-hydraulic actuator with a 500-kN capacity reacting on the large-capacity vertical reaction wall. The axial force (N=236kN - corresponding to 0.1 $A_g f_c'$) are applied by means of 6 hydraulic jacks, 2 per column (see Figure 4). The accelerogram used for tests 1 and 2 is compatible with the design requirements of the CHBDC 2000 for the region of moderate seismic activity in Eastern Canada with a return period of 475 years. The accelerograms used for the third and the fifth tests are compatible with the uniform hazard spectrum recommended by the NBCC 2005 for moderate seismic activity and high seismic activity regions located in eastern Canada respectively. The El Centro recording of the 1940 Imperial Valley earthquake is also used for the fourth test with peak ground acceleration scaled to 0.40g. Accelerograms 1, 2, 3 and 5 are uniform hazard spectrum-compatible time histories inputs (Atkinson and Beresnev, 1998). The initial stiffness for the tested substructure was determined by carrying out a static displacement test on the bridge bent specimen before each test. The bridge bent specimen was subjected to the 1st level of earthquake. Two of the three columns were then retrofitted with CFRPs and the bridge bent specimen was submitted to tests 2 to 5.

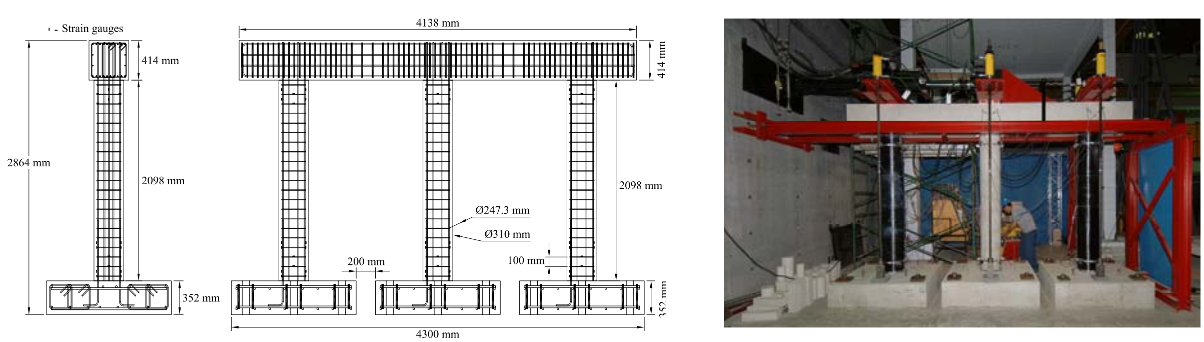

Figure 4. Reinforcement details and instrumentation and as-built bridge bent specimen.

Table 1. Experimental program.

Test	Details	Return period (years)	Seismic site activity	PGA (g)
1	CHBDC 2000, before retrofit	475	moderate	0.180
2	CHBDC 2000, after retrofit	475	moderate	0.180
3	NBCC 2005	2500	moderate	0.373
4	El Centro	-	-	0.400
5	NBCC 2005	2500	high	1.450

Test Results

The earthquake response of the bridge bent specimen, in terms of stiffness, top lateral displacement, base shear forces, displacement ductility and lateral drift values obtained during each test, is given in Table 2. The ductility reached a value of 3.01 after the 5th test, which is more than the ductility value of 2.80 calculated for retrofit design. During this high intensity test, the CFRPs showed no sign of distress and the strain values measured on the fibers were very low, showing that the design procedure is conservative. The behavior, the damage and the performance level are also given based on observations and measurements. These values correspond to the expected behavior of a normal bridge according to the performance matrix shown on Figure 1. Under the first two seismic inputs, the bridge bent specimen exhibited no visible cracking and performed with no yielding of the reinforcement. Under the seismic inputs of tests 3 and 4, yielding was measured on some longitudinal bars, but the center column of the bridge bent exhibited no significant cracking. Under the 3rd level of seismic intensity (test no 5), the center column exhibited more significant yielding and cracking, but no spalling of the concrete cover. These data can be referred to when establishing quantitative criteria for performance-based retrofit methods. It is also interesting to note that these values are similar to values given elsewhere for a ductile moment-resisting R/C frame (Ghobarah, 2004).

Table 2. Test results and damage levels.

Test	K (kN/mm)	u_{max} (mm)	V_{max} (kN)	μ_Δ	Drift (%)	Behavior	Damage level	Performance
1	18.0*	3.77	90.0	0.36	0.17	elastic	minor	immediate use
2	18.0	4.52	85.4	0.43	0.21	elastic	minor	immediate use
3	14.2	10.28	162.5	0.98	0.40	elastic limit	reparable	operational
4	13.2	12.38	174.8	1.18	0.50	elastic limit	reparable	operational
5	7.2	31.67	262.9	3.01	1.51	inelastic	major	life safe

*stiffness K before test : 26.0 kN/mm

Numerical Modeling and Dynamic Test

One of the main reasons for performing PSD tests is the difficulty to model the non-linear behavior of structures under seismic loading. Tests results are needed to validate numerical modeling assumptions. In this project, a three-dimensional finite element model was developed using the non-linear analysis program Ruaumoko to predict the behavior of the bridge. In order to closely represent the behavior of the actual bridge, dynamic tests were first carried out on site. Vertical and horizontal motions were recorded under ambient vibrations on both sides of the deck to allow for the detection of horizontal, vertical and torsional modes. Data were recorded with 6 velocity transducers placed evenly on the deck at 9-m intervals at a total of 22 locations. 6 modes were identified from the tests, 3 of them being transversal modes. Results of the on-site dynamic tests under ambient vibrations were used to update the numerical model. Once calibration was done in the linear domain, non-linear properties were assigned to appropriate members. A Rayleigh damping model proportional to the initial stiffness and mass matrices was used with a ratio of damping of 1.5% on the first two transverse modes. The effective properties for the columns as well as the cross-beams of the two bridge bents were obtained from sectional analysis performed with WMNPhi and a modified-Takeda hysteretic behavior was assumed in plastic hinges regions of the columns (Figure 5).

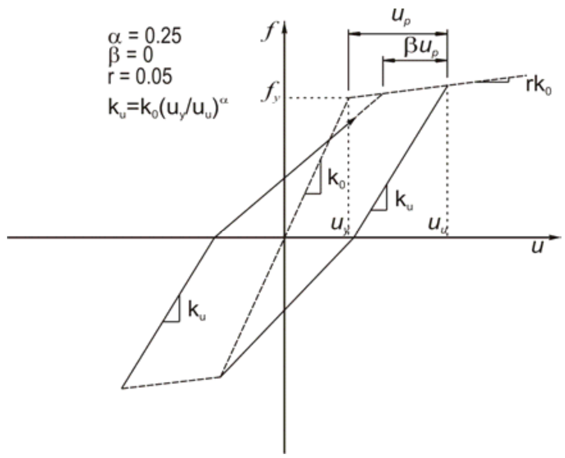

Figure 5. Modified-Takeda hysteretic model.

Table 3 shows a comparison between the measured responses from the PSD laboratory tests and the computed responses with Ruaumoko for the five tests. Note that stiffnesses of the columns were reduced between each test to account for cumulative damage. Figures 5, 6 and 7 show comparison between the measured and the computed responses of the bridge bent specimen for the three intensities ground motion. The graphics show the time history of the top lateral displacement of the bridge bent. Good agreement between the predicted and the measured responses can be observed according to the motion, the maximum top lateral displacement and the apparent period. More specifically, Figure 8 (test 5) demonstrates that the non-linear bridge bent behavior was very well predicted using the Ruaumoko and WMNPhi programs.

Table 3. Comparison between measured (PSD laboratory tests) and computed responses.

Test	**u_{max} (mm)**			**V_{max} (kN)**		
	Experimental	Ruaumoko	Error (%)	Experimental	Ruaumoko	Error (%)
1	3.77	4.14	+9.0	90.0	99.1	+10.1
2	4.52	4.60	+1.6	85.4	77.1	-9.7
3	10.28	10.66	+3.7	162.5	168.2	+3.5
4	12.38	12.62	+2.0	174.8	165.0	-5.6
5	31.67	29.39	-7.2	262.9	187.2	-28.8

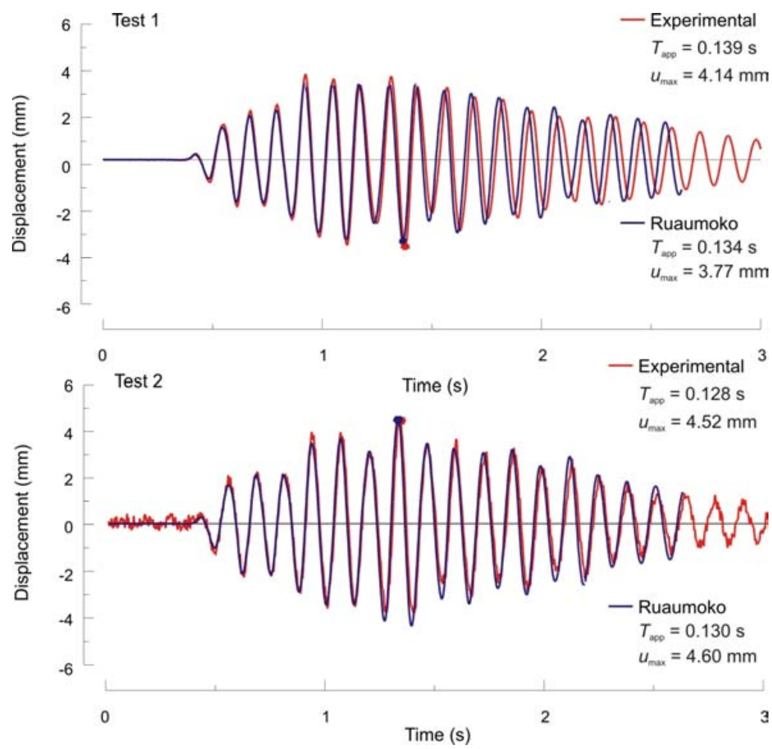

Figure 6. Intensity level 1: Comparison between measured and computed time history.

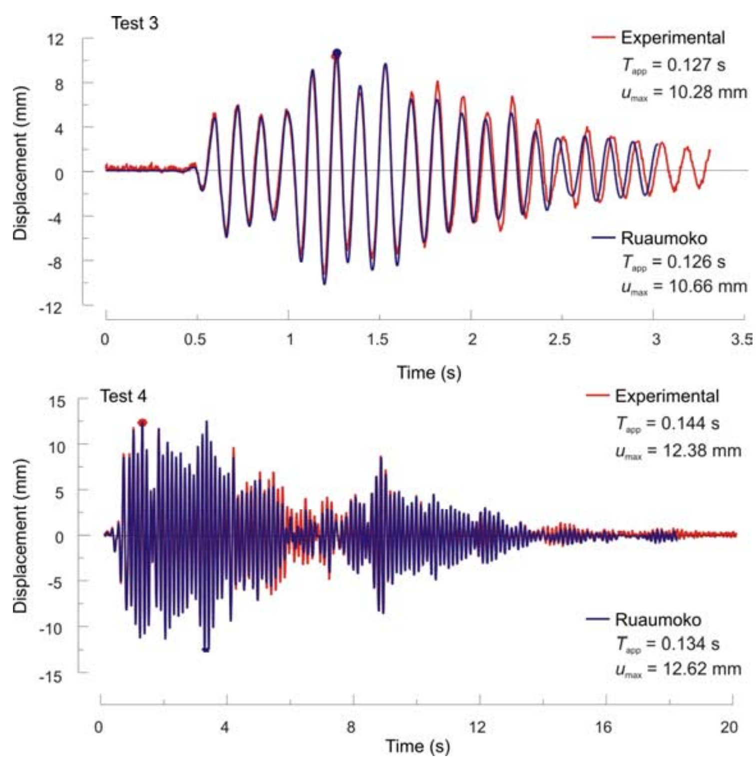

Figure 7. Intensity level 2: Comparison between measured and computed time history.

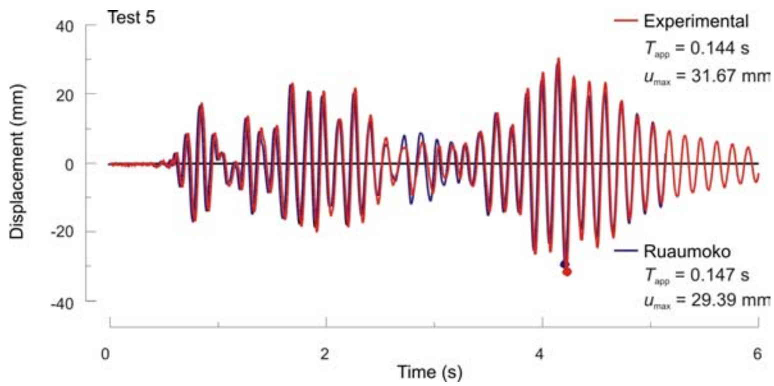

Figure 8. Intensity level 3: Comparison between measured and computed time history.

Conclusions

Recent changes in the 2005 edition of the National Building Code of Canada lead to an increase in earthquake hazard for some regions in Canada. In this project, a particular bridge bent seismic retrofit scheme was developed to study the impact of this increase. A large scale three-column bridge bent specimen was subjected to simulated earthquake loading using the pseudo-dynamic method with substructuring to evaluate the performance of the retrofitting methodology. This method, which represents the state-of-the-art in earthquake testing, supports best a performance-oriented test protocol considering input motions of increasing intensity corresponding to various limit-states of the bridge. During the highest intensity test, the CFRPs showed no sign of distress and the strain values measured on the fibers were very low showing that the design procedure is conservative. The data acquired during this series of test can be referred to when establishing quantitative criteria for performance based retrofit methods. Results of on-site dynamic test under ambient vibration were used to update a 3D finite element model of the bridge. This finite element model was developed with the non-linear analysis program Ruaumoko and the sectional analysis program WMNPhi. These tools proved to be effective in predicting the non-linear response of the bridge bent.

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