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A BRIDGE DUCTILITY STUDY FOR SEISMIC ASSESSMENT AND REHABILITATION

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SUMMARY

The main objective of this study is to compute a seismic vulnerability classification index that accounts for bridge bent ductility. With the help of our partners, the Quebec Ministry of Transportation and the city of Quebec, several structures were identified for this ductility study, and are representative of reinforced concrete bridges in eastern Canada. For each structure, the sectional ductility was first computed using a multi-layered sectional analysis program. Structural ductility values were then obtained with a simplified pushover analysis carried out with a specialized non-linear lumped-plasticity finite-element program. In a final step, the ductility demand was obtained from a time-history analysis, using six generated earthquakes for Eastern Canada. The ductility values were then used to propose a ductility-based evaluation criterion, to be used in a seismic vulnerability assessment method.

INTRODUCTION

Recent earthquakes in urban areas have demonstrated the vulnerability of some reinforced concrete structures to withstand seismic loads. Many failed structures, such as bridges, had originally been designed in accordance with older standard practices. Design codes have improved significantly over the years and, as a consequence, reparation and retrofitting have become necessary in order to extend the service life of many existing bridges constructed before seismic actions were adequately understood.

Since it would prove too expensive to retrofit each and every bridge, efficient rapid screening methods are needed. These methods should take into account such factors as seismic activity, hazard, social impact and structural vulnerability. Most methods are designed to take into account the different structural deficiencies, but the risk of a column failure is not well defined. An evaluation method, based on ductility demands has been developed and used to establish a priority list within a group of bridges in the eastern part of Canada.

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EVALUATION OF SEISMIC VULNERABILITY

It is recommended that the seismic vulnerability of bridges be defined using a holistic approach, taking account of the structural aspects, the seismic risks as well as the social and economical impacts of a bridge failure. Even though the probability of damage increases with the earthquake intensity and also depends on the soil conditions, the structural properties (geometry, materials, etc.) must be considered in a vulnerability analysis. The impact of closing down a bridge and its relative importance in the community should also be included.

In this section, evaluation procedures used in Japan, California and France are briefly reviewed. Methods that are being considered in Eastern Canada (Quebec) are then presented in more detail, as the selected bridges for this study are located in this region.

Bridge evaluation in Japan

The Japan Highway Public Corporation (JHPC) has developed a method to determine a list of bridge structures that should be retrofitted, following the Kobe earthquake in 1995. The main criteria that are considered in this method are:

- The economical impact of bridge failure;
- The year of construction (after 1980, codes and standards are better adapted);
- The type of bearings.

The Japanese Ministry of Construction (MOC) is using an approach developed by the Public Work Research Institute. Their evaluation procedure is based on structural criteria, such as the type of superstructure; the overall bridge geometry; materials used and their state of degradation; soil properties and condition; and the conditions of the bridge piers. The strategic importance of the bridge is not considered (Légeron, [1]).

Bridge evaluation in California

The California Department of Transportation (CALTRANS) developed a method where the risk is evaluated by multiplying a failure probability with its consequences (Gilbert, [2]). The procedure attributes more importance to the social and economical impacts (60%) than to the actual seismic vulnerability stemming from structural deficiencies (40%).

Bridge evaluation in France

The seismic behaviour of new bridges is considered in the AFPS 92 guide [3] and the Eurocode 8 [4]. An evaluation method is proposed by Conte [5], to guide owners in establishing priorities for seismic retrofitting. In this study, methods developed in the US by the Memphis and Shelby counties (Tennessee), by the US Department of Transportation and by the State of Washington, were adapted to the French context (a large number of bridge types found in France do not exist in the US). The proposed method is based on a global index computed from vulnerability criteria for the overall bridge, deck, piers, abutments and foundations.

Bridge evaluation in Eastern Canada (Quebec)

The Quebec Ministry of Transportation [6] uses three categories to characterize the state of its bridge structures. These categories are then weighed according to Table 1, and this leads to a combined index for a given bridge (ICS).

Table 1 – Combined index for a structure (ICS)

Categories	Description	Weight (%)
IFS	Structural function	65
IES	Structural condition	30
VS	Seismic vulnerability	5

Structural function index (IFS)
The structural function index is a combination of the following factors:

- FS: Strategic importance factor;
- FR: Road importance factor;
- FD: Detour importance, in the case of bridge closure;
- A function parameter based on computed bridge capacity; posted load limit; traffic volume; vertical and horizontal clearances; hydraulic behaviour, if applicable; state of the approaches; presence of pedestrian walkways and bicycle lane.

Structural condition index (IES)
This factor accounts for structural elements in the following categories:

- F: Foundation elements;
- S: Structural system ;
- P: Deck;
- ES: Secondary structural elements.

The following criteria are also considered for this index:

- FS: Strategic importance factor;
- FR: Road importance factor;
- FC: Cost of replacement factor.

Seismic vulnerability index (IVS)
This index was inspired by work by Filiatrault *et al* [7, 8] and is obtained by the following equation:

$$IVS = 100 - [(RS)(FF)(FA)(0{,}22C_1+0{,}22C_2+0{,}15C_3+0{,}13C_4+0{,}07C_5+0{,}02C_6+0{,}07C_7+0{,}12C_8)]$$

with the following factors:

- RS: Seismic risk factor;
- FF: Foundation factor;
- FA: Age factor (Table 2);
- C_i : Seismic influence factor (Table 3).

Table 2 – Age factor

Year of construction	Age factor (FA)
1990 – Today	0,7
1980 – 1989	0,8
1960 – 1979	0,9
Before 1959	1,0

Table 3 – Seismic influence factor

Factor	Description	Value (%)
C_1	Type of bridge	22
C_2	Complexity of structural behaviour	22
C_3	Number of discontinuities in superstructure	15
C_4	Vertical support element redundancy	13
C_5	Type of bearings	7
C_6	Bridge skew	2
C_7	Number of bridges	7
C_8	Public service	12

This method does not account for social and economical impacts when computing the seismic vulnerability index. Also, this index accounts for only 5% of the overall index (ICS).

It is important to note that in all the methods reviewed above, there is very little consideration for the bridge piers. These elements often have a great influence on the behaviour of the bridge under earthquake loading. One of the objectives of the research project presented herein is to propose an index that would account for the ductile behaviour of bridge piers and that could be included in a rapid screening method.

BRIDGES SELECTED FOR THIS STUDY

In order to evaluate the ductility demands and to propose a ductility-based evaluation index, seven reinforced concrete bridges were identified in the region under consideration, with the help of the Quebec Ministry of Transportation and the city of Quebec. These structures were selected based on their eligibility for a retrofitting program involving composite materials. The characteristics of the bridges (numbered S1 to S7) are presented in Tables 4 and 5. Other considerations for some bridges included geometry aspects related to laboratory tests, including regular geometry; absence of skew; circular piers and maximum pier height of 6m.

Table 4 – Selected bridges

Bridges	Year of construction	Number of bridge bents	Length (m)	Width (m)	Skew (degrees)
S1	1977	2	106,0	13,2	0
S2	1980	2	114,1	13,4	0
S3	1955	4	153,0	13,0	0
S4	1989	1	74,5	11,0	0
S5	1970	1	55,0	28,0	14
S6	1972	2	98,0	32,3	6
S7	1961	2	74,4	31,0	32

Table 5 - Bridge bents description

N^{O}	Number of columns	Height(m)	Diameter or width (mm)	Transverse reinforcement	Longitudinal reinforcement
S1	3	6,3	914	Ties No 5 s = 300 mm	15 No 11
S2	2	4,80	1220	Ties 10 mm s = 300 mm	32 No 35
S3	3	5,64	1220	Spirals 9 mm s = 50 mm	16 No25
S4	1	6,5	2100	Ties 15 mm s = 450 mm	50 No 30
S5	8	5,66	810	Ties No 3 s = 450 mm	12 No 9
S6	9	2,3	760	Spirals No 3 s = 57 mm	12 No 9
S7a	3	3,48	1524	Spirals s = 76 mm	18 No 11
S7b	3	2,438	1370	Spirals s = 76 mm	14 No 10

ANALYSIS PROCEDURE

In order to characterize the seismic vulnerability of the selected bridges, a ductility study of the bridge piers was carried out. Each structure was analysed using the following approaches:

- Sectional ductility evaluation;
- Structural ductility evaluation, using adaptive pushover analysis;
- Ductility-demand evaluation, using simplified non-linear time-history analysis.

Sectional Ductility

Each structure was modelled with the WMNPhi program [9]. This software, developed at the University of Sherbrooke, is used to predict the moment-curvature response using several stress-strain models. The bridge pier sections are modelled with several layers and the following algorithm is used:

- Assume the strain, e_{cc}, in the most compressed fiber;
- Assume the neutral axis position, c;
- Compute the resulting stresses in concrete and steel;
- Iterate on c until the resulting forces are in equilibrium;
- Compute moment, axial force and curvature - M, N and f;
- Increment the strain, e_{cc}, and repeat the process.

The moment-curvature responses were computed with an axial load corresponding to the non-factored gravity loads, including the superstructure and pier weights. Nominal material coefficients were used.

A stress-strain model developed by Légeron and Paultre [10] was used to model the confined concrete core. The stress-strain relationship shown in Figure 1 accounts for the progressive nature of the passive confinement. This curve is completely defined given two points : (1) the confined compressive strength, $f_{cc}^{'}$ corresponding to the strain $\varepsilon_{cc}^{'}$, and (2) the postpeak axial strain ε_{cc50} in the concrete when the capacity drops to 50% of the confined strength. The ascending branch of the stress-strain relationship of confined concrete is given by

$$f_{cc} = f_{cc}^{'}\left[\frac{k\left(\varepsilon_{cc}/\varepsilon_{cc}^{'}\right)}{k-1+\left(\varepsilon_{cc}/\varepsilon_{cc}^{'}\right)^{k}}\right], \quad \varepsilon_{cc} \le \varepsilon_{cc}^{'}$$

where the prime in a term indicates that it is evaluated at the peak of the stress-strain curve. f_{cc} is the stress in the confined concrete corresponding to strain ε_{cc} and k is a parameter controlling the slope of the ascending branch and is given by

$$k = \frac{E_{ct}}{E_{ct} - \left(f_{cc}^{'}/\varepsilon_{cc}^{'}\right)}$$

where E_{ct} = tangent modulus of elasticity of the unconfined concrete. The postpeak branch is given by

$$f_{c} = f_{cc}^{'}\exp\left[k_1\left(\varepsilon_{cc} - \varepsilon_{cc}^{'}\right)^{k_2}\right], \quad \text{for } \varepsilon_{cc} \ge \varepsilon_{cc}^{'}$$

where k_1 and k_2 are two parameters controlling the shape of the stress-strain curve. Based on experimental data, Légeron and Paultre [10] suggested the following equations :

$$k_1 = \frac{\ln 0.5}{\left(\varepsilon_{cc50} - \varepsilon_{cc}^{'}\right)^{k_2}}$$

$$k_2 = 1 + 25\left(I_{e50}\right)^2$$

where I_{e50} is the effective confinement index evaluated at the postpeak strain ε_{cc50}.

Steel stress-strain relationship includes strain hardening and is defined as in Park and Paulay [11]. Sectional ductility, μ_{φ}, is computed based on analysis carried out with WMNPhi and is defined as :

$$\mu_{\varphi} = \frac{\varphi_u}{\varphi_y}$$

where φ_u is the ultimate curvature and φ_y is the curvature at yielding.

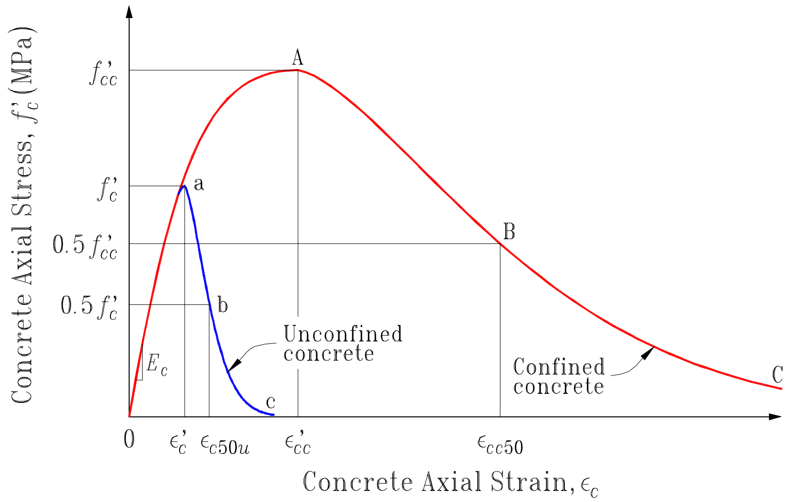

Figure 1 - Stress-strain relationship of confined concrete - Légeron and Paultre [10]

Simplified Structural Ductility

The overall structural ductility for each bridge was determined with an adaptive inelastic static analysis. The bridge piers were subjected to lateral loads causing successive yielding of the base and the top of the columns and subsequently their ultimate rotation. The lateral load and the resulting displacement are computed incrementally, as a function of the progressive yielding of the piers. The ultimate displacement is a function of the ultimate plastic rotation.

Each bridge bent was modelled with the RUAUMOKO program [12]. The beams were considered axially rigid. The plastic moment, M_y , the maximum axial capacity (compression and tension), as well as φ_u and φ_y were obtained from the sectional ductility analysis.

Ductility demand

The ductility demand was computed with a time-history analysis, using the RUAUMOKO program. It is designed to produce a time-history response of a non-linear general two-dimensional framed structure to ground acceleration. A bi-linear stress-strain relationship was used for the analysis. A post-yielding stiffness coefficient of 0.005 to 0.01 was used along with a viscous damping ratio of 3 %. The modified Takeda hysteresis model was used with an unloading stiffness coefficient of 0.25 and a reloading stiffness coefficient equal to 0.

Six earthquake records were used for the study and are described in Table 6. Earthquakes T1 to T4 are spectrum-compatible time-histories for 1/2500 per annum uniform hazard spectra (Adams *et al* [14]). These simulated time histories for eastern Canadian earthquakes of moment magnitude M6.0 and M7.0 at various distances, were generated by G. Atkinson [14] and were generously provided to the authors. The distances are indicated in the table (R values) and a tuning factor (S in the table) was used to match the target spectra for the bridge locations. Two moderate earthquakes were used to match the short-period hazard and two large distant earthquakes to represent the long-period hazard (Atkinson *et al* [14]). It is believed that these simulated records provide a realistic representation of ground motion for the earthquake magnitudes and distances that contribute most strongly to hazard at the selected region and probability level.

Records T5 and T6 were selected from a large number of synthetic motions generated with the SIMQKE [15] program. to match the 1/2500 p.a. uniform hazard spectra for bridge locations. These motions can be taught of as a composite of a number of events of different magnitudes at different distances. Record T5 was chosen from a set generated with an exponential envelope on the acceleration histories and record T6 was chosen from a set generated with a trapezoidal envelope as indicated in Table 6. The spectra for the generated records are compared in Figure 2 with the 1/2500 p.a.uniform hazard spectra for the one of the selected regions in Eastern Canada. Figure 3 presents the time histories of the generated records.

Table 6 – Description of earthquake records

Designation	Origin	PGA (g)	Duration (s.)	Description
T1	Adams *et al.*	0,37	8,88	M6 R = 30 km S = 0,85
T2	Adams *et al.*	0,44	8,88	M6 R = 30 km S = 0,85
T3	Adams *et al.*	0,27	21,07	M7 R = 70 km S = 0,90
T4	Adams *et al.*	0,26	21,07	M7 R = 70 km S = 0,90
T5	SIMQKE	0,27	15	Exponential
T6	SIMQKE	0,27	15	Trapezoidal

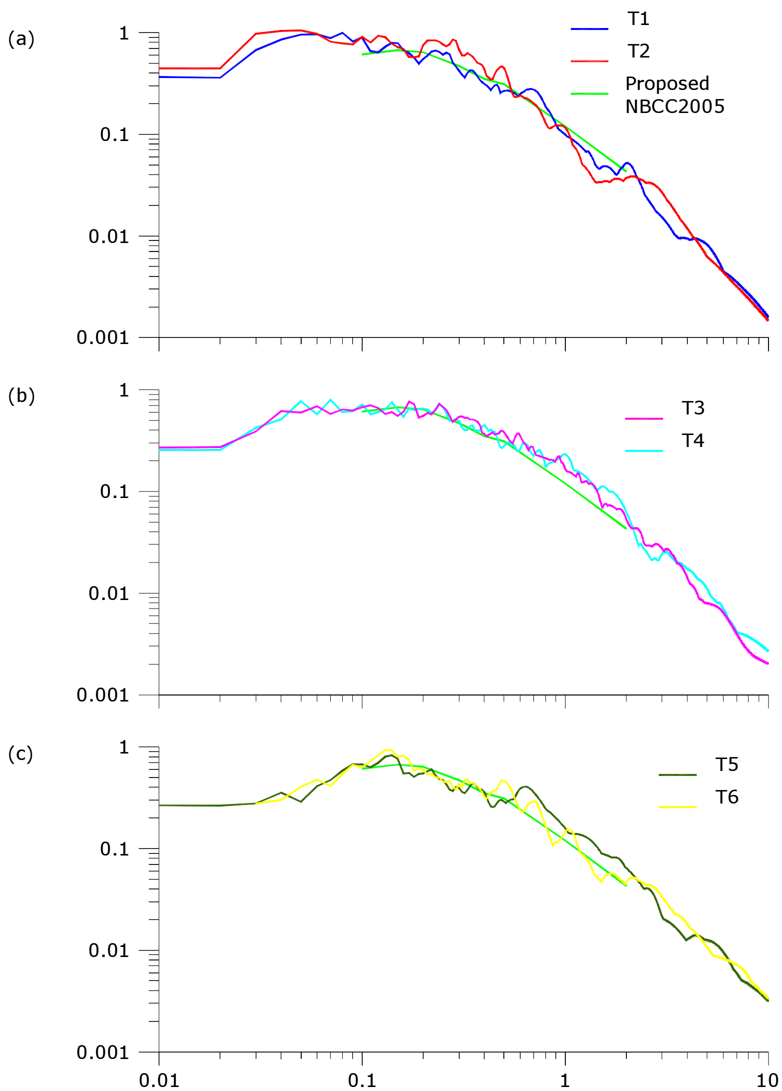

Figure 2– Uniform Hazard Spectra for a selected Eastern Canada site, according to the proposed 2005 edition of the National Building Code of Canada (Adams et al [13]), compared with spectra for records (a) T1, T2; (b) T3, T4; and (c) T5, T6.

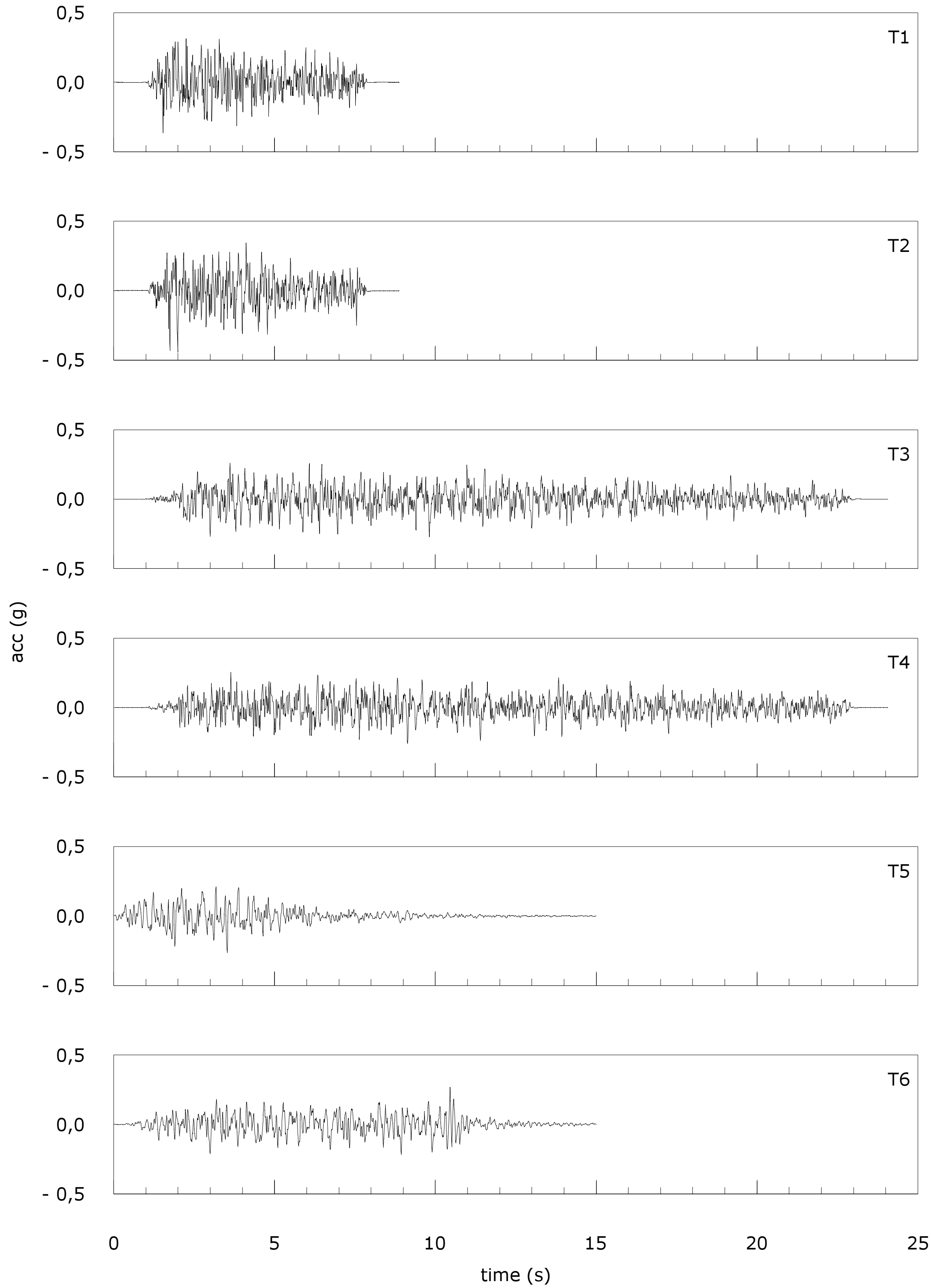

Figure 3 – Earthquake records used in this study

Results for a typical bridge

Results obtained for bridge S1 are presented below. Results for the sectional ductility obtained with the WMNPhi program and predicted with an adaptive pushover analysis with the RUAUMOKO program are shown in Figure 4. The force-displacement curve obtained with the pushover analysis is shown in Figure 5, together with a 3D view oft the modelled bridge bent.

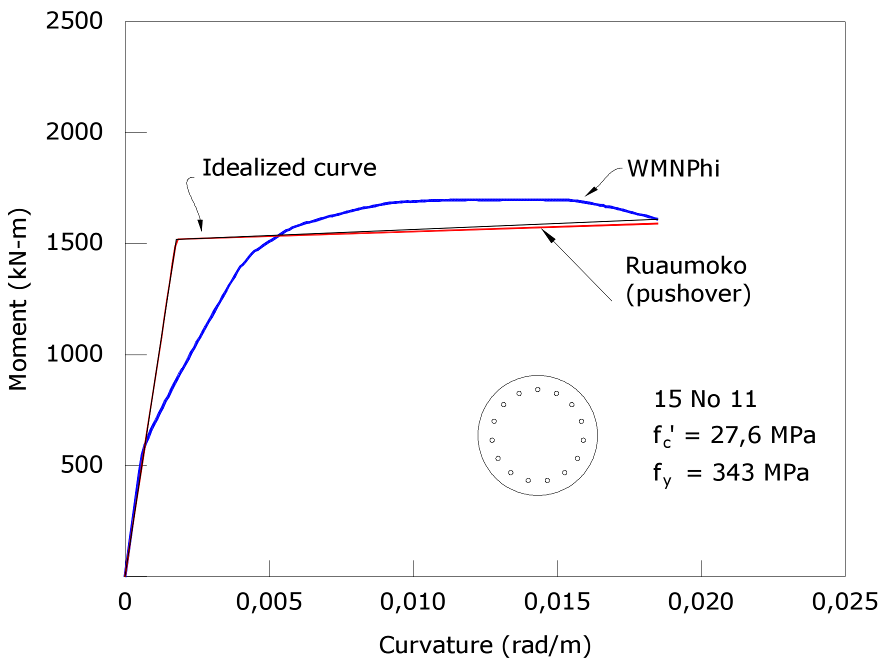

Figure 4 - S1 Moment-curvature

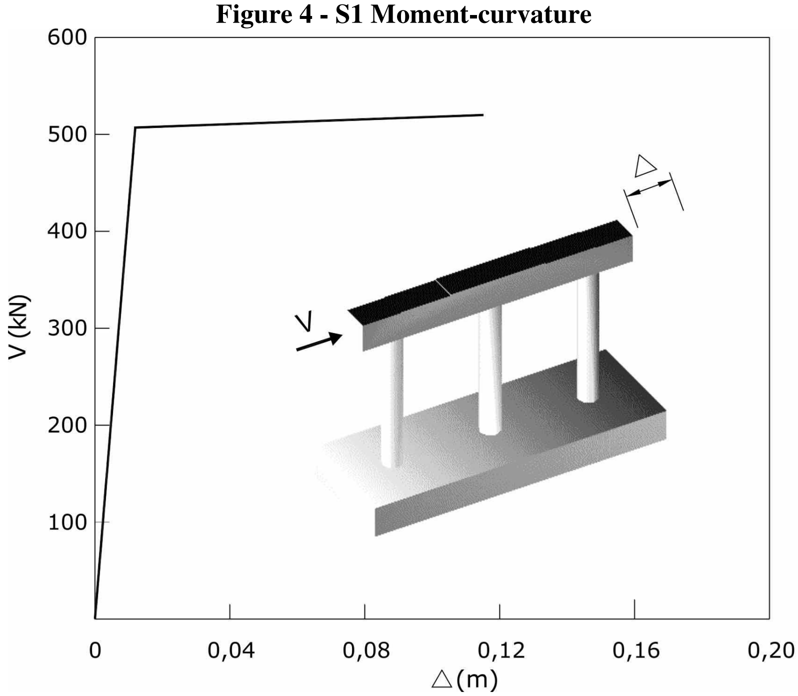

Figure 5 - Pushover and bridge bent for bridge S1

The hysteresis curves obtained for bridge S1 are shown in Figures 7 (a) to (f) for the six earthquake records that were used in this study.

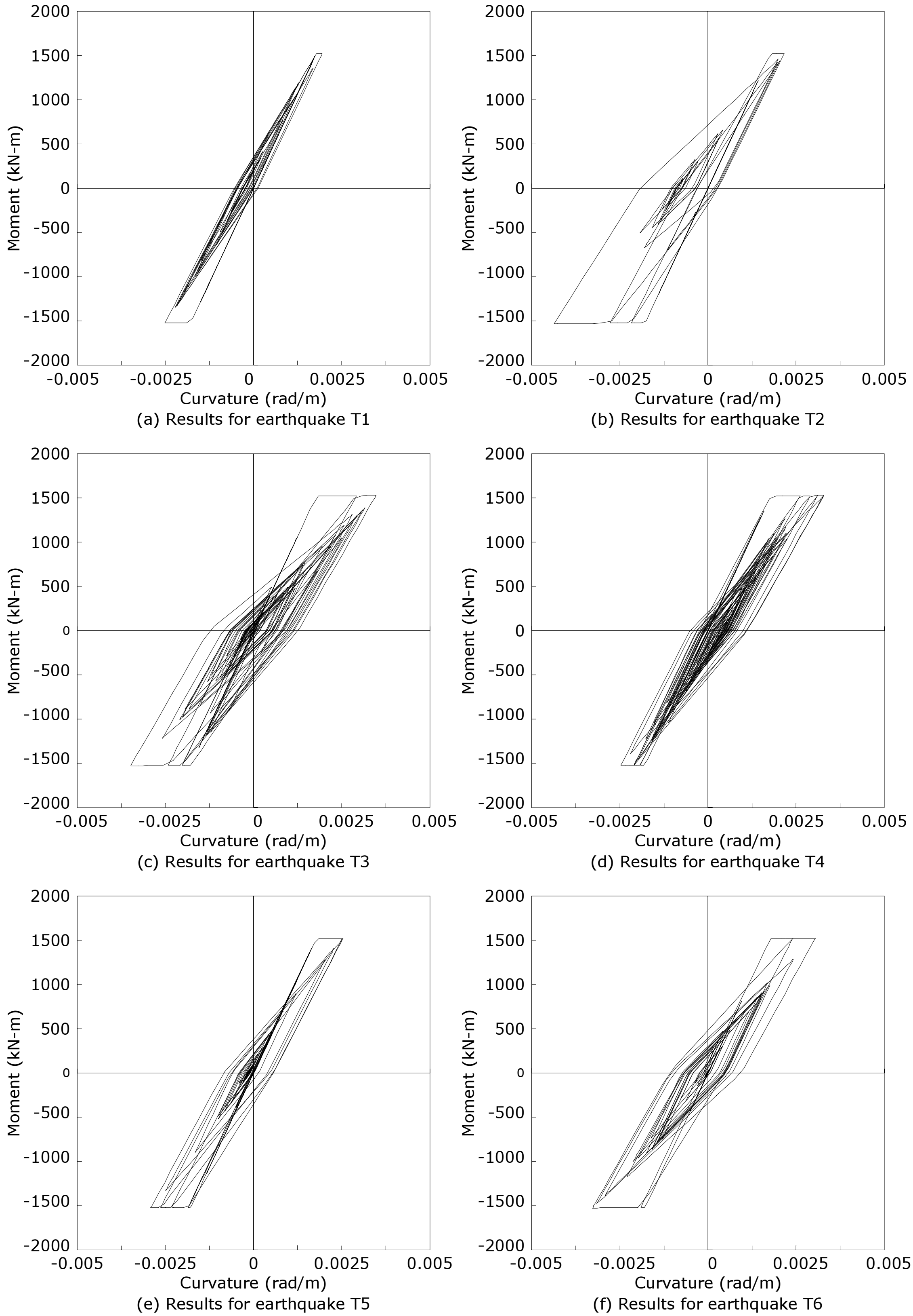

Figure 6 - Hysteresis curves for bridge S1

DISCUSSION

As expected, bridge S1 exhibits significant non linear response when subjected to all six earthquakes**.** As can be seen from Figure 6 c and d, nonlinear excursions are more pronounced for the larger magnitude and more distant events. A summary of the results obtained for the bridge ductility study is presented in tables 7 to 9. For each bridge bent, results for the sectional ductility, based on the moment-curvature analysis, are first presented in Table 7. The structural ductility results obtained from the pushover analysis, including the ultimate displacements (Δ_u), are presented in Table 8. Ductility and displacement demand (μ_d and Δ_d) obtained from the time-history analysis,, are presented in Table 9.

Table 7 –Sectional ductility from moment-curvature analysis

Bridge	φ_y (rad/m)	φ_u (rad/m)	μ_φ
S1	0,00185	0,018	9,72
S2	0,00461	0,012	2,60
S3	0,00450	0,016	3,55
S4	0,00250	0,007	2,76
S5	0,00649	0,012	1,85
S6	0,00625	0,015	2,40

Table 8 –Structural ductility from pushover analysis

Bridge	Δ_y (m)	Δ_u (m)	μ_Δ
S1	0,01190	0,1150	9,66
S2	0,01873	0,0723	3,86
S3	0,02586	0,0731	2,82
S4	0,03435	0,0425	1,23
S5	0,03480	0,0686	1,97
S6	0,00397	0,0147	3,70

Table 9 – Ductility demand from time-history analyses

Bridge	φ_d (rad/m)	μ_d	Δ_d (m)
S1	0,0033	1,77	0,02118
S2	0,0051	1,10	0,02005
S3	0,0057	1,28	0,02946
S4	0,0019	0,76	0,01340
S5	0,0079	1,22	0,04354
S6	0,0059	0,95	0,00753

Proposed ductility index

Using the results from the analysis carried out on each bridge, a proposed ductility ratio, R_μ, is presented in Table 10. It is obtained by dividing the ultimate displacement, Δ_u obtained from the pushover analysis, by the displacement demand, Δ_d, obtained from the time history analysis. A lower value for this index indicates a structure with a lower capacity/demand ratio.

It is important to note that this is a qualitative comparison of the ductility of bridge bents for a selection of typical bridges in the province of Quebec. This method has certain limitations, one of witch being that the shorter piers would probably exhibit shear failure and never reach the computed structural ductility. For example, bridge S7 has a low slenderness ratio (l/d) and would probably not fail in flexure, and bridge S6 is at the limit of failing in flexure. A shear behaviour study would need to be carried out to complement this ratio, but this was not one the objectives of the project.

In a seismic evaluation context – a retrofitting project for example – the proposed index is an interesting correlation between the ductility ratios and the earthquake behaviour of the bridge bents. It can be used to identify, within a given sample of bridges, the structures that are more vulnerable and that should be prioritized in a rehabilitation program. It could also be easily included in a classification procedure that would account for the structural behaviour of bridge piers.

Table 10 - Proposed ductility index, R_m

Bridge	Δ_u	Δ_d	$R_\mu = \Delta_u / \Delta_d$
S1	0,11500	0,02118	5,429
S2	0,07230	0,02005	3,606
S3	0,07310	0,02946	2,482
S4	0,04249	0,01340	3,171
S5	0,06863	0,04354	1,576
S6	0,01470	0,00750	1,953

CONCLUSION

A review of current seismic vulnerability evaluation methods and preliminary selection procedures put forth the need to consider the importance of bridge bents as energy dissipating mechanisms during earthquakes. In this regard, a sample of typical reinforced concrete bridges found in Eastern Canada was selected to carry out a ductility study. Sectional and structural ductilities were evaluated for each bridge pier. Simplified pushover and time-history analyses were carried out and the resulting ultimate displacement and ductility demands were used to compute a ductility index. The proposed index could be used in a seismic vulnerability screening procedure to identify, within a given group of bridges, the best candidates for rehabilitation. The ratio does not account for low slenderness ratios (short columns), and a more detailed analysis of the bridge bents shear behaviour should be completed to establish the applicability of the proposed index.

This research work is the basis for an experimental program, where rehabilitation techniques will be evaluated by means of sub-structure pseudo-dynamic testing of large-scale models of the bridge bents selected for this study. These tests will include an evaluation of the bridge bent behaviour before and after rehabilitation, as well as the computation of a damage index.

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