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THE USE OF PERFORMANCE BASED SEISMIC DESIGN FOR CANADIAN HIGHWAY BRIDGES

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ABSTRACT

In recent years there have been significant advances in the evaluation of seismic hazard and earthquake resistant design of structures. The developments of uniform hazard spectra (UHS), the increase in probability level in specifying seismic risks, the refinement of soil factors and the adoption of a performance based design approach in earthquake engineering have led to some significant changes in design practices. These changes have been incorporated in the 2003 Applied Technology Council (ATC) seismic design guidelines for highway bridges (ATC/MCEER 2003) and the 2005 edition of the National Building Code of Canada (NBCC 2005). The significance and impact of adopting the recent seismic hazard definition proposed by the Geological Survey of Canada (GSC) and National Earthquake Hazard Reduction Program (NEHRP) site factors on the design and performance of typical highway bridges located in Canadian cities of low, moderate and high seismicity are studied. The displacement ductility demand and rotational ductility demand are calculated to evaluate the performance of bridges using ground motions compatible with 10% in 50 year UHS and 2% in 50 year UHS seismic hazard values. A procedure for incorporating the seismic hazard definition adopted by the 2005 NBCC, refined site response factors and performance based objectives for Canadian Highway Bridges is presented.

Introduction

Recently, the National Building Code of Canada (NBCC) has been updated in its seismic provisions for design of buildings in Canada, including the adoption of seismic hazard specified by Uniform Hazard Spectra (UHS). The changes provide a more accurate method of delineating the effects of the overlying soil on the seismic effects on a structure and encourage the use of more detailed analysis and design methods. The latest edition of the Canadian Highway Bridge design code, CHBDC 2006, has seismic hazard specifications based on seismic hazard map information published by GSC in 1985, which is now obsolete. New seismic hazard maps of Canada based on improved seismicity models and recent earthquake data are now available (Adams and Halchuk, 2003). The seismic analysis and design methods specified in CHBDC 2006 are adapted from the specifications published by the American Association of State Highway and Transportation Officials (AASHTO) developed 20 years ago.

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Therefore, there is the need to evaluate the impact and significance of the recent developments and adoption of the UHS format of seismic hazard specification used in the 2005 NBCC on bridge design code practices in Canada.

In this study, the ductility of typical bridge structures common in Canada are evaluated by non-linear time history analysis, using ground motions compatible with the 10% and 2% in 50 year UHS seismic hazard values. Ground motions compatible with the short and long period range of the UHS are used to compare the effect of soil factors specified in 2005 NBCC on the seismic response of an example bridge structure.

The effects of recent changes in the seismic design approach of NBCC 2005 and ATC/MCEER 2003 on two Canadian highway bridges are studied. The analysis method in this study has been adapted from NBCC 2005 and ATC/MCEER 2003. The seismic hazard considered in this study is specified by following the procedure of 2005 NBCC to construct the UHS for a probability of exceedance 2% in 50 years. The seismic design and analysis requirements are determined using performance objectives defined in ATC/MCEER 2003. The life safety performance objective is chosen for the example bridges in this study. Seismic design and analysis procedure E, as proposed in ATC/MCEER 2003, is followed to analyze the bridges. The effects of soil types and factors specified in NBCC 2005 are studied. The reference ground condition (soil type C) is used for the first part of the study, while the effects of using time history ground motions matching the 2% in 50 year UHS for soil type A, B, C, D and E are studied in the second part of the study. The displacement and rotational ductility demands determined from the analysis are compared with the force modification factors (R factors) recommended by ATC/MCEER 2003. The plastic hinge length, required seat width and rotational capacity of the bridge columns are evaluated based on the seismic design guidelines proposed by ATC/MCEER 2003.

Development of UHS Compatible Ground Motions

Simulated Ground Motion records compatible to the 10% and 2% in 50 year UHS (Atkinson, 1998) are used as input excitation in the bridge performance study here. For each probability level three different suites of ground motion are provided, one for eastern Canada, one for western Canada and one for the Cascadia earthquake scenario. The 10% in 50 year set of ground motions includes two trials for each magnitude distance combination and the 2% in 50 year set of ground motions includes four trials for each magnitude distance combination. In this study, the ground motions for eastern Canada are adjusted to match the UHS seismic hazard in Montreal, Ottawa and Toronto, while the western Canada ground motions are adjusted to match the UHS seismic hazard in Vancouver. Two time history records are used for the bridge performance study in each selected city, one time history record representative of an earthquake event of moderate magnitude at closer distance with the characteristic of higher seismic energy in the short period range (0.1-0.5s) and another representative of an earthquake of larger magnitude at a greater distance with the characteristic of higher seismic energy in the long period range (0.5 –5s). The time history records are scaled to best fit the target UHS values. The appropriate scaling factors used in this study are determined within the range 0.5 to 2 in increments of 0.1 (Atkinson et al., 1998) by minimizing the variance between the selected simulated earthquake time history record and the target UHS (Phung, 2005).

Tables 1 and 2 show the magnitude, distance, scale factor, maximum acceleration (unscaled) and duration of the ground motions matching the UHS for each city for 10% and 2% in 50 year probability of exceedance in the short and long period ranges. The characteristics of the ground motions matching the 10% in 50 year and 2% in 50 year UHS in Montreal and Ottawa are the same. In the short period range for 10% and 2% in 50 year probability of exceedance, the same ground motion recording is used by applying different scaling factors for Montreal/Ottawa and Toronto. The ground motions for use in Vancouver show larger energy input than the ground motions for eastern Canada.

Table 1. Ground Motion records matching 10% in 50 year UHS

10% -50years	Montreal		Ottawa		Toronto		Vancouver	
Period Range	$T \leq 0.5s$	T>0.5s	$T \leq 0.5s$	T>0.5s	$T \leq 0.5s$	T>0.5s	$T \leq 0.5s$	T>0.5s
Magnitude	5.5	7.0	5.5	7.0	5.5	7.0	6.0	7.2
Distance (km)	70	300	70	300	70	300	20	50
Scale Factor	2.0	0.8	2.0	0.8	0.8	0.5	0.9	1.0
A_{gmax} cm/s ^{2})	47.34	42.82	47.34	42.82	47.34	53.52	206.3	146.66
Duration (s)	15.5	32.07	15.5	32.07	15.5	32.07	6	17.98

Table 2. Ground Motion records matching 2% in 50 year UHS

2% - 50years	Montreal		Ottawa		Toronto		Vancouver	
Period Range	$T \leq 0.5s$	T>0.5s	$T \leq 0.5s$	T>0.5s	$T \leq 0.5s$	T>0.5s	$T \leq 0.5s$	T>0.5s
Magnitude	6	7.0	6	7.0	6	7.0	6.5	7.2
Distance(km)	50	50	50	50	50	100	30	40
Scale Factor	1.5	0.6	1.5	0.6	0.6	0.5	1.0	0.5
A_{gmax} (cm/s ^{2})	198.15	619.76	198.15	619.76	198.15	237.81	526.83	496.02
Duration (s)	12.4	20.55	12.4	20.55	12.4	23.07	8.53	16.64

Example Bridges

Highway bridges are critical links in the Canadian transportation system. In Ontario the 400 series highways form the most important transportation network in the province. Two common types of bridges in Ontario are chosen for this study. The first is the Barnsdale Road Underpass, which is a two-lane bridge passing over the 416 Highway south of Ottawa. The second is the CNR Overhead EBL, which carries Highway 417 east bound lanes over the CNR easement in eastern Ottawa. The force-deformation relationship of the bridge columns has been determined using a section fiber model and the BIAX program (Wallace, 1992). The mode shapes and frequencies of the structures have been determined using the NEABS program (Penzien et al., 1981). The displacement, rotational and curvature ductility are determined from dynamic time-history analysis using the NEABS program. The analyses have been performed for Montreal, Ottawa, Toronto and Vancouver using ground motions compatible with the 10% in 50 year and 2% in 50 year UHS representative of earthquake events in the short and long period range.

Barnsdale road Underpass is a two span, single column, prestressed bridge. The column is monolithic to the bride deck and the bridge has laminated elastomeric bearings and expansion joints at each abutment and a piled foundation. The total length of the bridge including abutments is 75 meters; the total length of the bridge from the center line of the abutment bearings is 64 meters. The height of the column from the top of the pile cap to the deck is 7.28 meters. The column is 1500 mm in diameter with 30- #35 longitudinal bars and a #15 spiral at 50 mm pitch. The concrete cover is 80 +/-20 mm. The deck is 9460 mm wide and 1250 mm deep at the center. Between the abutments and the column a hollow concrete section is used, while at the abutments and over the column the section is solid. The modified Kent and Park model is used to model the stress-strain behaviour of the concrete (Park et. al., 1982). A bilinear approximation of the stress-strain relation is adopted to determine the yield curvature (Priestly et al., 1996). The yield curvature is found to be $\phi_y = 0.00215$ (/m). An elevation view of the Barnsdale road underpass is given in Fig. 1, and the bridge cross section properties are presented in Table 3.

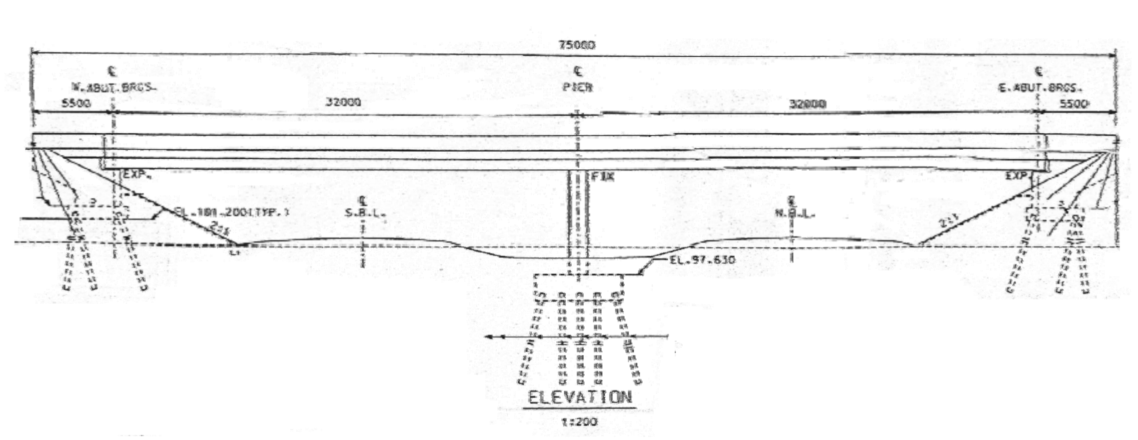

Figure 1. Elevation View Barnsdale Road Overpass

Table 3. Barnsdale Road Overpass cross section properties.

Element type	area m^2	inertia x m^4	inertia y m^4	inertia z m^4
Deck Section 1	8.378	3.549	1.492	19.872
Deck Section 2	10.852	3.768	1.316	28.39
Column Section	1.767	0.754	0.377	0.377

The CNR Overpass is a five span, multi-column, prestressed girder bridge. Two similar structures are located parallel to each other to separately carry east and west bound traffic. The bridge carrying east bound lanes is chosen for the study. The bridge spans at abutments are supported on bearings. There are expansion joints at the abutments and the two exterior piers, while the two central piers are fixed to the superstructure. The bridge has a piled foundation. The total length of the bridge including abutments is 94 meters, the total length of the bridge from the center line of the abutment bearings is 72 meters. The two exterior spans are 11.2 meters in length, and the three interior spans are each 16.8 meters in length (from the center line of the abutment bearings). The bridge superstructure is constructed of seven AASHTO II prestressed girders, continuous over the superstructure. The width of the bridge varies from 14 meters to 18 meters along its length from north to south. The girders are rigidly connected to each other by transverse diaphragms at the abutments and pier supports. The bridge is supported on four multi-column bents and each bent has three columns. The columns are rigidly connected to a pier cap. The columns at piers #1 and #4 adjacent to the north and south abutments respectively are approximately 4.25 meters in length. At piers #1 and #4 the bridge superstructure is supported on the pier cap by elastomeric bearings. The columns at pier #2 and #3 are approximately 8 meters in length. At piers #2 and #3 the bridge superstructure is supported on the pier cap by elastomeric bearings and the pier cap and superstructure are connected using dowels. The columns of piers #2 and #3 are connected by a collision strut, which is located approximately 3.6 meters above the pile cap. The pier columns are 914 mm in diameter with 12- 35mm longitudinal bars and a 15mm spiral at 50 mm pitch. The concrete cover is 76 mm. The modified Kent and Park model is used to model the stress-strain behaviour of the concrete (Park et. al., 1982). A bilinear approximation of the stress-strain relationship is adopted to determine the yield curvature (Priestly et al., 1996). The yield curvature is found to be ϕ_y =0.004 (/m). An elevation view of the CNR Overpass is given in Fig. 2 and the bridge cross section properties are presented in Table 4.

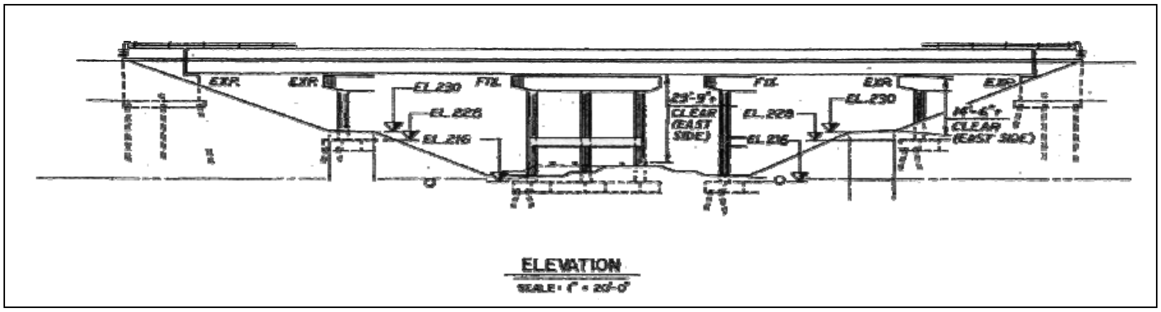

Figure 2. Elevation View CNR Overhead EBL

Table 4. CNR Overhead EBL cross section properties.

Element type	area m^2	inertia x m^4	inertia y m^4	inertia z m^4
Deck Section 1	4.695	66.6	0.554	65.5
Deck Section 2	15.706	268.6	1.637	267
Pier cap	1.28	0.228	0.11	0.169
collision strut	0.348	0.015	0.017	0.006
column section	0.656	0.069	0.034	0.034

Dynamic Time History Analysis

The impact and significance of recent seismic design developments and adoption of the UHS format of seismic hazard specification in the 2005 NBCC on the bridge design practices in Canada are evaluated by means of dynamic time history analysis. The purpose of this analysis is to obtain insights in light of the recent developments in earthquake engineering design that will lead to better performance objectives and safer and more efficient seismic design of highway bridges in Canada.

Non-linear time history analyses have been performed on the Barnsdale Road Underpass and the CNR Overpass using the NEABS program to determine the responses of the structures to ground motions compatible to the 10% and 2% in 50 year UHS for the reference ground condition. The 2% in 50 year UHS spectra for soil types A to E for Montreal/Ottawa, Toronto and Vancouver have been constructed by multiplying the UHS for the reference ground condition by the appropriate F_a and F_v factors of NBCC 2005. Ground motions compatible with the short and long period UHS are used to compare the effect of the 2005 NBCC soil factors on the seismic response of Barnsdale Road Underpass. The seismic responses and performance of the Barnsdale Road Underpass and the CNR Overpass are assessed by means of the displacement and rotation requirements of ATC/MCEER 2003.

Yield Displacement and Rotation

The yield displacement is calculated using the formula (Priestley et al., 1996):

$$\Delta_y = \frac{\phi_y l^2}{3} \tag{1}$$

$$\mu_\Delta = \frac{\Delta_{\max}}{\Delta_y} \tag{2}$$

with the assumption of either single or double curvature, where ϕ_Y is found from the moment curvature relationship. Displacement capacity verification (push-over) analysis to determine the yield displacement is performed by recording the top of column displacement at the first yield of any column member. The yield displacement found by the push-over analysis results are selected to calculate the displacement ductility factor. The rotational ductility factors have been calculated using the yield curvature from the moment-curvature analysis multiplied by the length of the section under consideration. The maximum rotational ductility demand is found by dividing the maximum rotation by the yield rotation as follows,

$$\theta_y = \phi_y l_p \tag{3}$$

$$\mu_\theta = \frac{\theta_{\max}}{\theta_y} \tag{4}$$

where l_p is the plastic hinge length or the length of the column section under consideration.

Response Modification Factors

The displacement and rotational ductility demand are related to the force reduction factor or the base response modification factor, R. The recommended R factor in CHBDC 2000 is 3.0 for single columns and the value is 5.0 for multiple columns bents. The recommended base response modification factor used with seismic design and analysis procedure E in ATC/MCEER 2003 is 6.0 for single and multiple columns. CALTRANS seismic design criteria 2004 recommend ductility demands less than or equal to 4.0 for single columns bents on fixed foundation and 5.0 for multi-column bents on fixed or pinned foundations. The displacement and rotational ductility demand calculated are compared to 6.0, as recommended by ATC/MCEER 2003 because the analysis procedure used in this study is similar to seismic design and analysis procedure E recommended by ATC/MCEER. The base response modification factor, R for Montreal/Ottawa, Toronto and Vancouver soil types A to E, 2% in 50 year UHS have been calculated using the formula,

$$R = 1 + (R_b - 1)\left(\frac{T}{1.25T_s}\right) \le R_b \tag{5}$$

where:

- R_b is equal to 6.0, for substructures consisting of multiple column bents;
- T_s=S_{dl}/$S_{ds,}$;
- S_{dl} =F_vS_l where S_l is the spectral acceleration corresponding to 1.0 seconds and F_v is the soil response factor in the long period range;
- S_{ds} =F_aS_s. where S_s is the spectral acceleration corresponding to 0.2 seconds and F_s is the soil response factor in the short period range.

The calculated value of R is greater than 6.0 for all cases, except Vancouver soil type E, where the value of R is 4.29.

Design Displacement and Rotation

The seat width and plastic rotation capacity of the Barnsdale Road Underpass and the CNR Overpass are calculated using ATC/MCEER 2003 recommended procedures. The calculated values are compared to the time history analysis results. The calculated required seat width is compared to the maximum time history displacement response, and the calculated plastic rotation capacity is compared to the maximum time history rotation response results. The Plastic hinge length and maximum seat width requirements of the example bridges are calculated by following the ATC 2003 method. The plastic hinge length of the Barnsdale Road Underpass and the CNR Overpass are determined by the one sixth the column length criterion, which is 1.21 meters for Barnsdale Road Underpass and 1.33 for the CNR Overpass. The minimum seat width for the Barnsdale Road Underpass and the CNR Overpass is governed by the criterion that the maximum seat width is taken as 1.5 times the maximum displacement from time history analysis. In Table 5 and 6 the design seat width is compared to 1.5 times the maximum displacement from time history analysis for Barnsdale Road and the CNR Overhead respectively, for the 10% and 2% in 50 year hazard levels. The plastic rotational capacity of the Barnsdale Road Underpass and the CNR Overpass are calculated, using the ATC/MCEER method. The total rotational capacity is then found by adding the plastic rotation capacity to the yield rotation determined from the moment-curvature relation. The maximum rotation from the time history analysis is compared to the calculated total rotational capacity. The results are presented in Table 5 and 6 for the Barnsdale Road underpass and the CNR Overhead respectively, for the 10% and 2% in 50 year hazard levels.

Analysis of Results

The seismic response in the transverse direction is less than that in the longitudinal direction for both structures and all load cases. The longitudinal response of the Barnsdale Road Underpass and CNR Overpass are observed to be non-linear when subjected to the Montreal/Ottawa seismic hazards of short and long period ground motions at the risk level of 2% in 50 year and the Vancouver seismic hazards at both the10% in 50 year and 2% in 50 risk levels.

Comparing the calculated base response modification factor of the example bridges with the time history ductility results, the calculated displacement and rotational ductility values from the time history analysis results are under 6 for all cases except for the case of Vancouver for the 2% in 50 year short period compatible ground motion, which has been found to be 6.9. The largest response, in terms of ductility demand were from the time history analysis are the Montreal/Ottawa 2% in 50 year long period ground motion and the Vancouver 2% in 50 year short and long period ground motion.

The required seat width has been calculated and compared to the maximum response displacement times 1.5, as recommended for design practice in ATC/MCEER 2003. The calculated required seat width ranged from 426mm to 578mm and is larger than 1.5 times the maximum response displacement in all cases. The seat width provided is 900mm for Barnsdale Road Underpass and 685mm for the CNR Overpass.

ATC/MCEER 2003 guidelines set limits on the plastic rotation capacity of columns. The total allowable rotational capacity should be the minimum of the calculated value or 0.035 plus the yield rotation. The maximum column rotation is found to be less than the minimum value for all cases. The results are presented in Table 5 and 6, for Montreal/Ottawa, Toronto and Vancouver, for both the 10% and 2% in 50 year hazard level and short and long period compatible ground motions.

The maximum displacement and rotation response of the Barnsdale Road Underpass to the 2% in 50 year UHS for soil types A to E in Montreal/Ottawa, Toronto and Vancouver are determined. The displacement ductility demand for the Ottawa long period ground motions for soil type D and E and for Vancouver short period soil type B, C, D and E and the long period soil type B and E are found to be greater than the recommended value. The rotational ductility demand for the Ottawa long period ground motions for soil type D and E are found to be greater than 6. In the case of Montreal/Ottawa and Toronto the increase in the 2% in 50 year UHS for soil types D and E creates response demands, which are greater than the recommended values. In the case of Vancouver the 2% in 50 year UHS for soil types B, D, D and E create response demands which exceed the recommended values. Fig. 3 and 4 present the displacement and rotational ductility for Montreal/Ottawa, Toronto and Vancouver as a function of the soil type.

Table 5. Barnsdale Road Underpass Seat Width and Rotation Capacity-Reference Ground Condition Site Class C

	Period Range	Hazard Level in 50 years	ATC/ MCEER calculated Seat Width (m)	Time History Analysis 1.5Δ (m)	ATC/MCEER calculated plastic rotational capacity of hinges θ_p	Total rotational capacity of hinges $\theta_y + \theta_p$	Time History Analysis Results θ_t
Montreal/Ottawa	<0.5s	10%	0.426	-0.00585	0.031	0.034	0.00071
Montreal/Ottawa	>0.5s	10%	0.426	0.0063	0.033	0.036	0.00064
Montreal/Ottawa	<0.5s	2%	0.471	0.0207	0.039	0.042	-0.0012
Montreal/Ottawa	>0.5s	2%	0.471	-0.0513	0.029	0.031	0.00525
Toronto	<0.5s	10%	0.411	0.0207	0.033	0.035	0.0003
Toronto	>0.5s	10%	0.411	-0.00255	0.034	0.036	0.00058
Toronto	<0.5s	2%	0.428	0.00495	0.034	0.037	0.00068
Toronto	>0.5s	2%	0.428	0.00645	0.029	0.031	0.00122
Vancouver	<0.5s	10%	0.491	0.00671	0.022	0.024	0.0011
Vancouver	>0.5s	10%	0.491	0.0162	0.023	0.026	0.00148
Vancouver	<0.5s	2%	0.566	0.0159	0.017	0.02	0.00991
Vancouver	>0.5s	2%	0.566	-0.0819	0.021	0.023	0.00438

Table 6. CNR Overhead EBL Seat Width and Rotation Capacity-Reference Ground Condition Site Class C

	Period Range	Hazard Level in 50 years	ATC/ MCEER calculated Seat Width (m)	Time History Analysis 1.5Δ (m)	ATC/MCEER calculated plastic rotational capacity of hinges θ_p	Total rotational capacity of hinges $\theta_y + \theta_p$	Time History Analysis Results θ_t
Montreal/Ottawa	<0.5s	10%	0.436	0.01125	0.053	0.058	-0.0013
Montreal/Ottawa	>0.5s	10%	0.436	0.0159	0.057	0.062	-0.0018
Montreal/Ottawa	<0.5s	2%	0.481	-0.033	0.067	0.072	0.0032
Montreal/Ottawa	>0.5s	2%	0.481	-0.0825	0.049	0.054	0.0235
Toronto	<0.5s	10%	0.42	-0.03	0.056	0.061	-0.0005
Toronto	>0.5s	10%	0.42	0.0045	0.058	0.063	0.0007
Toronto	<0.5s	2%	0.437	-0.006	0.058	0.063	0.0016
Toronto	>0.5s	2%	0.437	-0.0135	0.049	0.054	0.0028
Vancouver	<0.5s	10%	0.501	0.0278	0.037	0.042	0.0405
Vancouver	>0.5s	10%	0.501	0.0441	0.04	0.045	0.0084
Vancouver	<0.5s	2%	0.578	-0.051	0.03	0.035	0.0184
Vancouver	>0.5s	2%	0.578	-0.1575	0.036	0.041	0.0062

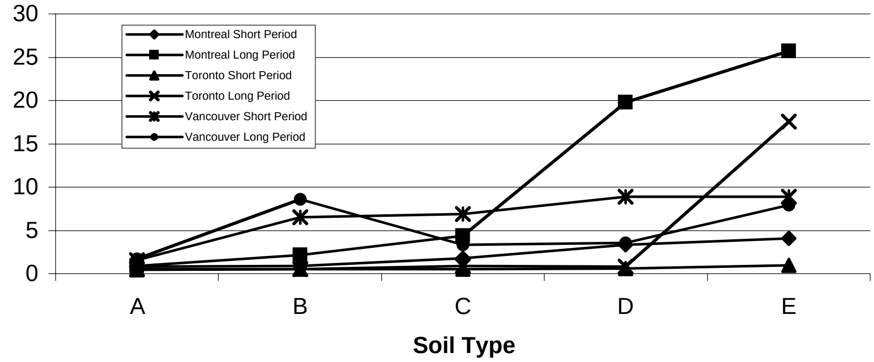

Figure 3. 2% in 50 year Displacement Ductility Versus Soil Type

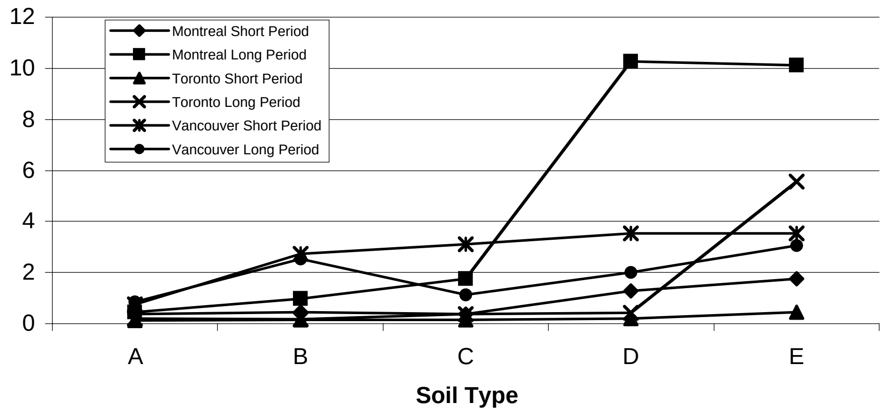

Figure 4. 2% in 50 year Rotational Ductility Versus Soil Type

Conclusion

The effects of recent changes in the seismic design methods of NBCC 2005 and ATC/MCEER 2003 on the seismic performance of two Canadian highway bridges have been studied. The seismic hazard is defined by the 2% in 50 year UHS in accordance to the specifications in NBCC 2005. Seismic design and analysis procedure E, as proposed in ATC/MCEER 2003 has been used to analyze the bridges. The life safety performance objective and the effect of adopting the soil types and factors in NBCC 2005 have been studied. The reference ground condition (soil type C) is used for the first stage of the analysis, while the effects of UHS matching ground motions corresponding to soil type A, B, C, D and E are studied in the second stage of the analysis. The plastic hinge length, required seat width and rotational capacity of the bridge columns are evaluated based on the seismic design guidelines proposed by ATC/MCEER 2003. The displacement and rotational ductility demands determined from the analysis are compared to force modification factors (R factors) recommended by ATC/MCEER 2003.

The results show that the two Ontario highway bridges studied have adequate seat width and rotational capacity when compared to the 2003 ATC/MCEER LRFD guidelines for the seismic design of highway bridges. However the displacement and rotational ductility demands, particularly when considering soft soil types often exceed the recommended value given in ATC/MCEER.

For further studies, it is recommended that the displacement and rotational ductility demands from non-linear analysis should be confirmed by more detailed investigations or laboratory testing, to determine whether these structures can develop the required ductility. The displacement and rotational ductility capacity of Canadian Highway bridges should be confirmed, for both the life safety and operational performance objectives. The target ductility values given in ATC/MCEER need to be evaluated to determine whether they are appropriate for the target reliability level and performance of Canadian Highway Bridges.

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