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Critical Review of the Elastic Force Factor in the 2019 Canadian Highway Bridge Design Code

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ABSTRACT

When elastic design forces are significantly lower than the forces derived from capacity-design principles, the 2019 Canadian Highway Bridge Design Code (CSA S6:19) permits using unreduced elastic force demands multiplied by 1.25 as design forces to protect against brittle failure modes. In a recent paper by the authors, it was shown that the margin provided by the 1.25 Elastic Force Factor could be inadequate to safeguard against brittle failure modes when structural components designed for seismic demands under the 5th generation seismic hazard values are subjected to the increased seismic demands from the 6th generation seismic hazard values. The present paper continues the previous research work with the main objectives of providing an in-depth review of the Elastic Force Factor in CSA S6:19 and identifying and investigating the parameters affecting the adequacy of the Elastic Force Factor to safeguard against undesirable failure modes. It is discussed whether the Elastic Force Factor should be a function of importance category and seismic performance category of the bridge. The effects of parameters such as site location, site soil classification, return period and type of earthquake ground motion, and various sources of uncertainty in demand and capacity are discussed and showcased through three case studies of a model reinforced concrete bridge in Victoria, BC. The performance of the bridge in each case is evaluated using nonlinear time-history analysis. It is demonstrated and verified further that the current value of 1.25 for the EFF in CSA S6:19 could be inadequate to safeguard against brittle failures and plastic hinge formation in undesirable locations.

Keywords: Elastic Force Factor, Elastic Design, Capacity-Design, Canadian Highway Bridge Design Code, 6th generation seismic hazard values.

INTRODUCTION

The Canadian Highway Bridge Design Code, CSA S6 [1-2], employs capacity-design as a design method to prevent seismic damage to certain components by making them strong enough to resist the seismic loads generated or transferred by the adjacent ductile or force-limiting components. Ideally capacity-protected components are designed to meet the maximum force effects that can be developed by the ductile elements attaining their probable resistances as part of a plastic mechanism (in the case of a fusing mechanism, capacity-design forces are the maximum forces that can be transferred by the fusing component). Probable resistances in CSA S6 are determined by multiplying resistances calculated using expected material properties by a factor of 1.15-1.3 depending on the seismic performance category of the bridge. While designing to maximum force effects corresponding to probable resistances (probable demands) is the preferred approach for capacity-design, in some cases where elastic design forces are significantly smaller than the probable demands, it might not be economical to do so. For such cases, CSA S6 allows using factored elastic demands for the design of capacity-protected component. The factor applied to the elastic seismic demands for the design of capacity-protected components is referred to as Elastic Force Factor (EFF).

How CSA S6 defines elastic seismic design forces for the design of capacity-protected components have changed over the past few cycles of the code. In CSA S6-14, seismic design forces for the design of capacity-protected components were unreduced elastic forces determined using response modification factor of R=1.0 and importance factor of I_E=1.0. Therefore, the EFF was equal to unity.

The 2019 edition of the code, however, specifies 1.25 times of the unreduced elastic seismic forces as the design force to prevent brittle failure modes. Similar provisions were specified earlier for the BC Ministry of Transportation and Infrastructure Supplement to CSA S6-14 [3]. The 25% increase in the elastic demands combined with the margin provided through factoring of resistances, was intended to ensure enough margin against premature brittle failure modes, where the components were not designed to the maximum probable demands. The 1.25 value for the EFF was selected rather arbitrarily based on the increase previously considered in the code for the elastic seismic design forces of connectors, and otherwise had no solid basis.

Recently, the BC MoTI Supplement to CSA S6:19 [4] updated its provisions to specify the elastic force factor as a function of the importance category of bridge with the value of 1.25, 1.35, and 1.5 for other, major-route, and lifeline bridges, respectively. The intent of using a larger Elastic Force Factor was to provide a larger safety margin for more important bridges, considering the significant increase in seismic hazard values from the 5th generation seismic hazard values prepared for the 2015 National Building Code of Canada [5] to the 6th generation seismic hazard values prepared for NBC 2020 [6. Similar to the 1.25 EFF in CSA S6:19, the newly updated EFF values in the BC Supplement to CSA S6:19 seem to be selected based on engineering judgment without an in-depth investigation of whether the specified values would indeed provide adequate margin against brittle failure modes. It is also not clear that aside from importance category, what other parameters should be taken into account when determining the EFF value.

In a recent paper by the authors [7], it was shown that the margin provided by the 1.25 EFF could be insufficient to safeguard against brittle failure modes in capacity-protected components. In that study, the performance of the case study bridges was evaluated using response-spectrum analysis and verified further by inelastic static pushover analysis. It was also shown that the increase in seismic demands of structural components when subjected to the NBC 2020 design spectrum, could be predicted by the ratio of the NBC 2020 spectral displacement to the NBC 2015 spectral displacement at the respective fundamental period of the bridge. The spectral displacement ratio needs to be calculated separately for each of the principal axes of the bridge.

The present paper builds upon the previous research work with two main objectives: firstly, to provide a critical review of the EFF in CSA S6 and to identify what parameters may affect the adequacy of the EFF value to safeguard against brittle failure modes in capacity-protected components; and secondly to investigate the effect of some of the identified parameters on the EFF value through a number of case studies. The performance of the case study bridges will be evaluated using nonlinear time-history analysis (NTHA) to accurately capture the effect of any nonlinearity in the system on the demands.

EXAMINATION OF THE ELASTIC FORCE FACTOR IN CSA S6

Elastic Force Factor in the Context of Capacity-Design

It is essential to recognize that the main purpose of the Elastic Force Factor in CSA S6 is for allowing the engineer to use a "proxy design approach" to capacity-design when achieving an economical design for capacity-protected components is not feasible through the standard capacity-design process. While the proxy design approach using Elastic Force Factor is applicable to structures responding primarily within the elastic range, it not exclusive to an elastic design process, and can be used in combination with any design approach including displacement-based design, force-based design, and isolation-based design.

The proxy design approach to capacity design is specified under Clause 4.4.10.4.2.2 of CSA S6:19 "seismic design forces for capacity-protected elements for force-based and performance-based design" and Clause 4.4.10.4.3 "yielding mechanism and design forces in ductile substructures." There is an inherent conflict in the provisions of the above-mentioned clauses, which is due to introducing the proxy design approach under the umbrella of capacity-design. While using factored elastic demands in place of probable demands provides some margin against brittle failure modes, inherently it does not provide full protection as achieved by capacity-design. Furthermore, capacity-design is based on establishing a clear hierarchy of strength between ductile and non-ductile components in a structural system to limit seismic damage to certain regions of the ductile (or fusing) components and preventing damage to other non-ductile components. This would require a full understanding of the desirable plastic (or otherwise fusing) mechanisms in the system and designing the ductile and capacity-protected component accordingly. In contrast, the proxy design approach in S6 does not necessarily require establishing a clear strength hierarchy in the system. As such, it would not be clear from the design process, which components in the system are expected to form a plastic hinge and in what order, if the system was subjected to larger seismic demands than the design seismic demands. Because of the above reasons, the term "capacity-protected" is not truly applicable to components designed using the proxy design approach and a more appropriate term such as "partially protected" or "marginally protected" should be used instead.

Another difference between capacity-design and the proxy design approach, is that the focus of the former is solely on protecting against brittle failure modes such as shear failure. Only the demands related to brittle failure modes are factored using the EFF and other demands including the bending moments for marginally protected components are unreduced elastic demands. This is while when performing capacity-design, flexural capacities of the capacity-protected elements are also set based on probable bending moment demands transferred from plastic hinging in ductile elements (or from a fusing mechanism). It can be argued that if the EFF was also applied to the flexural capacities of the marginally protected components, a hierarchy of strength would have been established (to some degree) in the system. In other words, in addition to brittle failure modes, the EFF could also provide margin against other undesirable failure mechanisms in the system, such as forming plastic hinges in undesirable locations.

Adequacy of the Elastic Force Factor

As mentioned earlier, in a recent paper by the authors [7] it was shown that the 1.25 EFF could be inadequate to safeguard against brittle failure modes in marginally protected components, considering the significant increase in the NBC 2020 seismic hazard values compared to NBC 2015 values. The increase in the NBC 2020 seismic hazard values varies depending on bridge site location, average shear wave velocity measured by V_{s30} of the site, vibration period, and ground motion return period. As an example, Figure 1 shows the ratio of the NBC 2020 to NBC 2015 spectral acceleration values for Site Class B, C, and D in Victoria, BC and for two hazard levels with 2% and 10% probabilities of exceedance in 50 years (2475-yr and 475-yr return periods, denoted as 2%/50 and 10%/50 in the figure). It can be readily observed that the increase in hazard values is larger for softer soils of Site Class D condition compared to Site Class B and C. The increase is also period dependent. For instance, for Site Class D there is an 86% increase[1] at the period of 2.0 s as opposed to a lower increase of 45% at 10.0 s. This figure shows that for all three site classes and at both hazard levels, the increase in seismic hazard values is generally larger than 25%. Such a significant increase in seismic loading implies that structures designed for seismic demands under the NBC 2015 seismic hazard values using the 1.25 EFF would very likely be deficient against brittle failure modes if subjected to the NBC 2020 seismic demands.

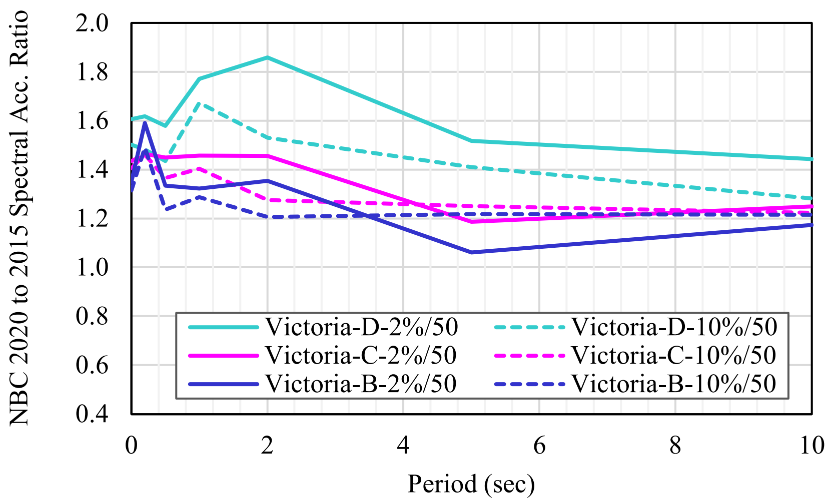

Figure 1. Ratio of the NBC 2020 to NBC 2015 uniform hazard spectrum spectral acceleration values for Victoria, BC

The necessary condition for the adequacy of the EFF is that it is calibrated such that all sources of uncertainty in the system, which would affect the demands and capacities of the marginally protected components are addressed. The changes in seismic hazard values reflect the uncertainty in seismic demands due to seismic hazard modelling. There are other sources of uncertainty that needs to be considered, including:

- Epistemic uncertainty in demand due to inaccuracies in defining modelling parameters such as yield strength, viscous damping, foundation flexibility, etc.
- Aleatory uncertainty in demand due to record-to-record variability, etc.
- Epistemic uncertainty in capacity prediction equations and bias in the test data, etc.
- Aleatory uncertainty in capacity due to natural variability of the material strength, construction tolerances, etc.

In a thorough examination of the Demand and Capacity Factored Design [8] as a potential probabilistic framework for CSA S6-14 performance-based Design, Ashtari [9] showed that the demand and capacity factors calculated assuming the recommended values in ASCE 7-16 [10] for the uncertainties in demand and capacity, can be in the order of 1.2 and 0.6, respectively (equivalent to a combined factor of 2.0). This means that even if the demands are increased by 25% using the EFF, a much larger reduction in capacity is necessary to address all the sources of uncertainty adequately. Further research is required to update the EFF taking into account the effect of all sources of uncertainty.

[1] The significant increase in site Class D values around 1.0-2.0 s period is a subject of debate and is being examined for validity.

PARAMETERS AFFECTING THE ELASTIC FORCE FACTOR

The Elastic Force Factor is essentially a demand factor applied to the elastic seismic demands of marginally protected components. The objective of the proxy design approach, as in any design process, is to ensure that adequate capacities are provided to meet the demands considering all sources of uncertainties. In the simplest form, the design equation can be written as follows:

$$EFF * D \leq \varphi\, C \tag{1}$$

In the above equation D and C are demand and capacity and φ is the capacity factor ($\varphi \leq 1$). Demand factors are generally a function of the following:

- Epistemic and aleatory uncertainties in demand; and
- Acceptable probability of failure (i.e., target reliability for the design)

In case of the EFF in CSA S6:19 however, the sources of uncertainty on the capacity side, which need to be considered in the capacity factor are not fully addressed. For reinforced concrete members, factored resistances in CSA S6:19 are calculated using material factors that consider only the material strength variability (the equivalent φ is about 0.9) and for steel members, the resistance factors for flexure, shear, compression, tension, and torsion ranges between 0.95 to 0.9. The capacity factors in both cases seem to be relatively insufficient to address all the sources of uncertainty on the capacity side for seismic design. Therefore, the EFF would inevitably become a function of the epistemic and aleatory uncertainties not currently covered with the material and resistance factors in CSA S6:19.

The recent increase in seismic hazard values from NBC 2015 to NBC 2020 can be categorized primarily under epistemic uncertainty in demand related to ground motion models. In this case, since both set of seismic hazard values are published, the change in the values can be quantified deterministically (and this is the approach taken so far in the previous and present research work by the authors). However, the change in seismic hazard is unknown for future seismic hazard model and should be quantified probabilistically for various ground motion models. Several parameters affect the uncertainty in seismic hazard and therefore may affect the EFF value, including:

- Bridge site location;
- Site soil classification and average shear wave velocity over the first 30 meter of soil (V_{s30});
- Ground motion return period
- Type of earthquake (crustal, subduction intraslab, subduction interface)
- Period of vibration.

The effect of some of the above parameters on seismic hazard values was shown earlier in Figure 1. The effect of the site soil class, ground motion return period, and type of earthquake will be explored further in the case studies.

The EEF is also a function of the assumed target reliability for the design. CSA S6:19 is not calibrated specifically to meet a target reliability factor for seismic design and as such, any assumptions around the target reliability for calibrating the EFF would be inconsistent with the rest of the seismic provisions in CSA S6. As explained earlier, the BC MoTI Supplement to CSA S6:19 has recently updated the EFF values to be a function of bridge importance category, with the value of 1.25, 1.35, and 1.5 for other, major-route, and lifeline bridges, respectively. It appears this update has been done with the intention of achieving higher reliability against brittle failure modes for more important bridges. However, the updated values are selected primarily based on engineering judgement rather than setting clear target reliabilities, which would require further studies. In addition, one could argue that the EFF should also be a function of Seismic Performance Category. In lower seismicity regions, as reflected by a lower Seismic Performance Category, the design of bridge components is often governed by load cases other than seismic. As such, the marginally protected components, oftentimes have considerable reserve capacities under seismic demands, which could provide extra margin against undesirable failure modes in addition to that provided by the 1.25 EFF.

CASE STUDIES

Description

The sensitivity of the EFF value to some of the identified parameters in the previous section is examined through three case studies. The bridge employed for all three cases is the same and is designed to remain essentially elastic under the NBC 2015 design spectrum for Victoria, BC, Site Class D condition with 2% probability of exceedance in 50 years. The bridge is designed according to CSA S6:19 provisions and using 1.25 EFF with elastic seismic demands calculated from Response Spectrum Analysis (RSA), according to standard practice. The performance of the bridge is then evaluated using nonlinear time-history analysis for the three cases listed in Table 1. The parameters changing between the cases include site soil class condition (Site Class B and D), seismic hazard level (2%/50 and 10%/50), and seismic hazard model (NBC 2015 and NBC 2020).

Table 1. Description of the case studies

Case Study	Site	Site Soil Class	Target Seismic Hazard Spectra	No. of NTHA
1	Victoria, BC	D	NBC 2015 & NBC 2020 at 2%/50	24
2	Victoria, BC	B	NBC 2015 & NBC 2020 at 2%/50	24
3	Victoria, BC	D	NBC 2015 & NBC 2020 at 10%/50	24

General Arrangement of the Bridge

The case study bridge is a two-span reinforced concrete bridge with composite steel plate girder concrete deck on piled foundations in Victoria, BC. Each span is 36 m long and 27 m wide. The deck is comprised of seven 1.5 m deep steel plate I-girders composite with a 225 mm thick concrete deck. The superstructure of the bridge is designed for the standard CSA S6:19 load combinations. A typical cross-section of the bridge deck is shown in Figure 2.

The steel girders are supported at the middle on a pier bent comprised of four 6 m long circular reinforced concrete columns and a 2 m wide 1.5 m deep pier cap. The pier columns are supported on a 6 m wide, 24 m long, 1.5 m deep pile cap with 2x8-1.22 m dia. concrete-filled steel pipe piles. At the abutments, the girders are supported on pile bents each comprising of 7-0.914 m dia. concrete-filled steel pipe piles. As mentioned in the description, the proportioning and design of the pier columns, piles, pier cap, and abutment pile caps were carried out for the demands from RSA such that they remain essentially elastic under the NBC 2015 2%/50 Site Class D design spectrum for Victoria. The marginally protected components were designed for 1.25 times of the elastic demand. Because the spectral acceleration values for the Site Class D design spectrum are larger than the Site Class B spectrum and so are the corresponding seismic demands, the bridge design was assumed adequate for Site Class B condition. An elevation view of the bridge is shown in Figure 3 and a summary of the section proportion is listed in Table 2.

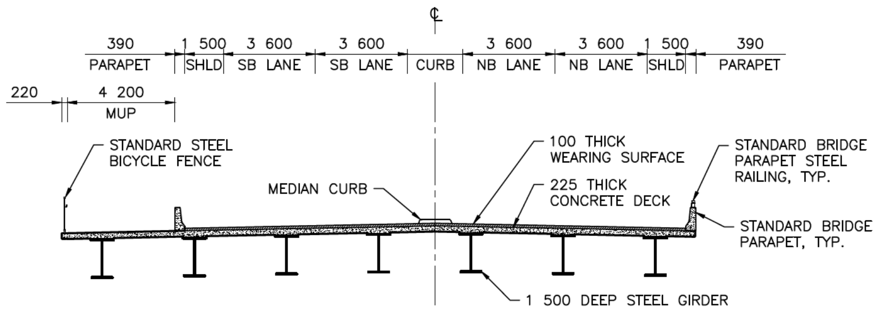

Figure 2. Typical cross-section of the case study bridge

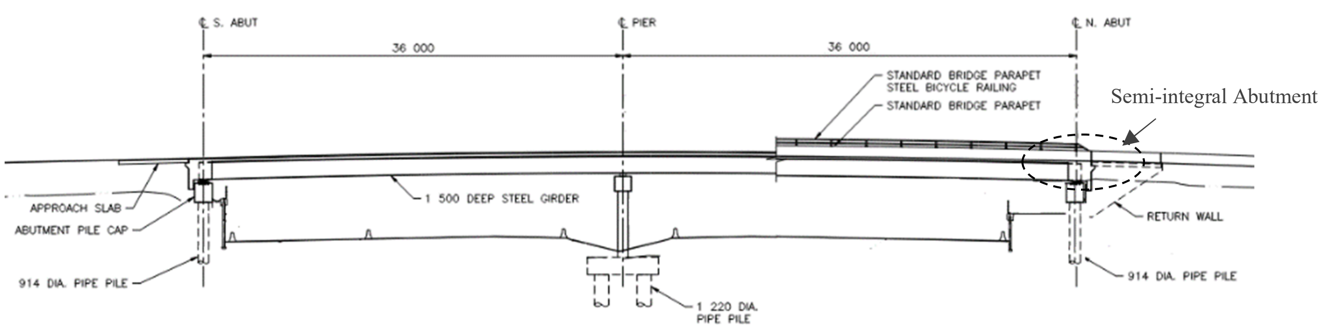

Figure 3. Elevation view of the case study bridge

Table 2. Section proportioning for the case studies

Component	Section
Columns	1500 mm dia., ρ=3.0 %
Pier Piles	1220 mm dia. Steel pipe pile, ρ=1.9 %
Abutment Piles	914 mm dia. Steel pipe pile, ρ=1.9 %
Pier Cap	1500x2000 mm
Pile Cap	1500x1500 mm

Site Soil Properties

To study the effect of site soil properties on the EFF value, two different site soil classifications were assumed for the three case studies. The site soil class is assumed to be Site Class D with V_{s30} of 270 m/s for the first and third case studies and Site Class B with V_{s30} of 1100 m/s for the second case study with no risk of liquefaction.

Bridge Analysis Model

A finite element model of the bridge was generated in CSI SAP2000 to perform NTHA. A grillage model was utilized for the superstructure and the substructure elements and piles were modelled using frame elements. For ductile elements and top of the abutment piles, effective section properties were assigned to modify the elastic stiffnesses for flexure and shear. Fiber interacting P-M2-M3 plastic hinges were assigned to potential plastic hinge locations including top and bottom of pier columns, top of the abutment and pier piles, ends of the pier cap segments between the columns, and ends of the pile cap segments between the piles. A mass and stiffness proportional Rayleigh damping was assumed for the NTHA.

The pier pile cap was modelled using thick shell elements. The depth-to-fixity of the piles were assumed to be five times their respective diameter. The soil-structure interaction at the semi-integral abutments was modelled using nonlinear abutment springs recommended by Caltrans Seismic Design Criteria 2.0 [11]. A snapshot of the bridge model is shown in Figure 4.

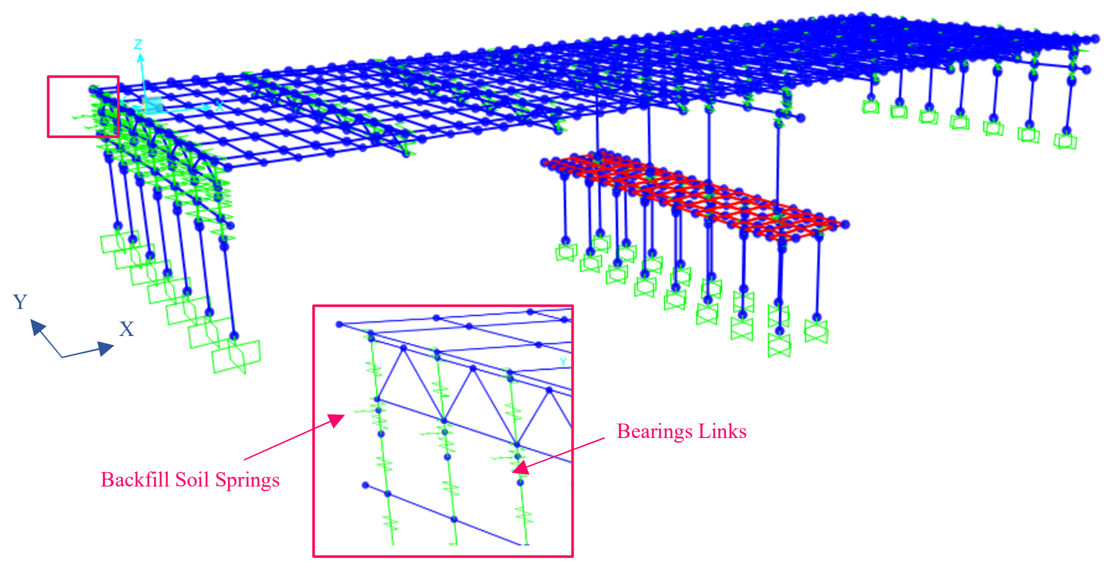

Figure 4. Bridge model in CSI SAP2000

Ground Motion Time Histories

The selection and scaling of ground motion time histories were carried out according to CSA S6:19 and the commentary to CSA S6:19 [12]. The commentary of CSA S6:19 recommends a minimum of 11 ground motions each containing two horizontal components to be selected for NTHA. The selected motions should be representative of the tectonic regime, magnitude and distances that control the seismic hazard, and the site condition. For each site soil class condition and each seismic hazard model, a suite of 12 ground motion time histories including 6 crustal motions and 6 subduction motions were selected and linearly matched to the target spectrum at 2%/50 hazard level by minimizing the mean squared error (MSE) within the recommended period range of interest. For the period range of interest, the commentary recommends a period range of $0.2T_1$ to larger of $2\ T_1$ and 1.5 s, where T_1 is the fundamental period of the bridge. For case study 1 and 3, T_1 was 0.7 s and 0.3 s in the longitudinal and transverse directions, respectively, and for case study 2, 0.45 s and 0.29 s. The range of the magnitude and site-to-sources distances of earthquakes with the highest contribution to the seismic hazard within the period range of interest were determined by deaggregation of the 2%/50 uniform hazard spectrum. At period 0.5 s, crustal earthquakes with magnitude range of 6.4-7.4 and site-to-source distance of 25-75 km and subduction interface with magnitude range of 8.5-9.0 and site-to-source distance of 50-90 km have the highest contribution to the 2%/50 Victoria UHS. Figure 5 shows the mean of the 12 ground motion records against the respective design spectrum for each of the selected suites of record.

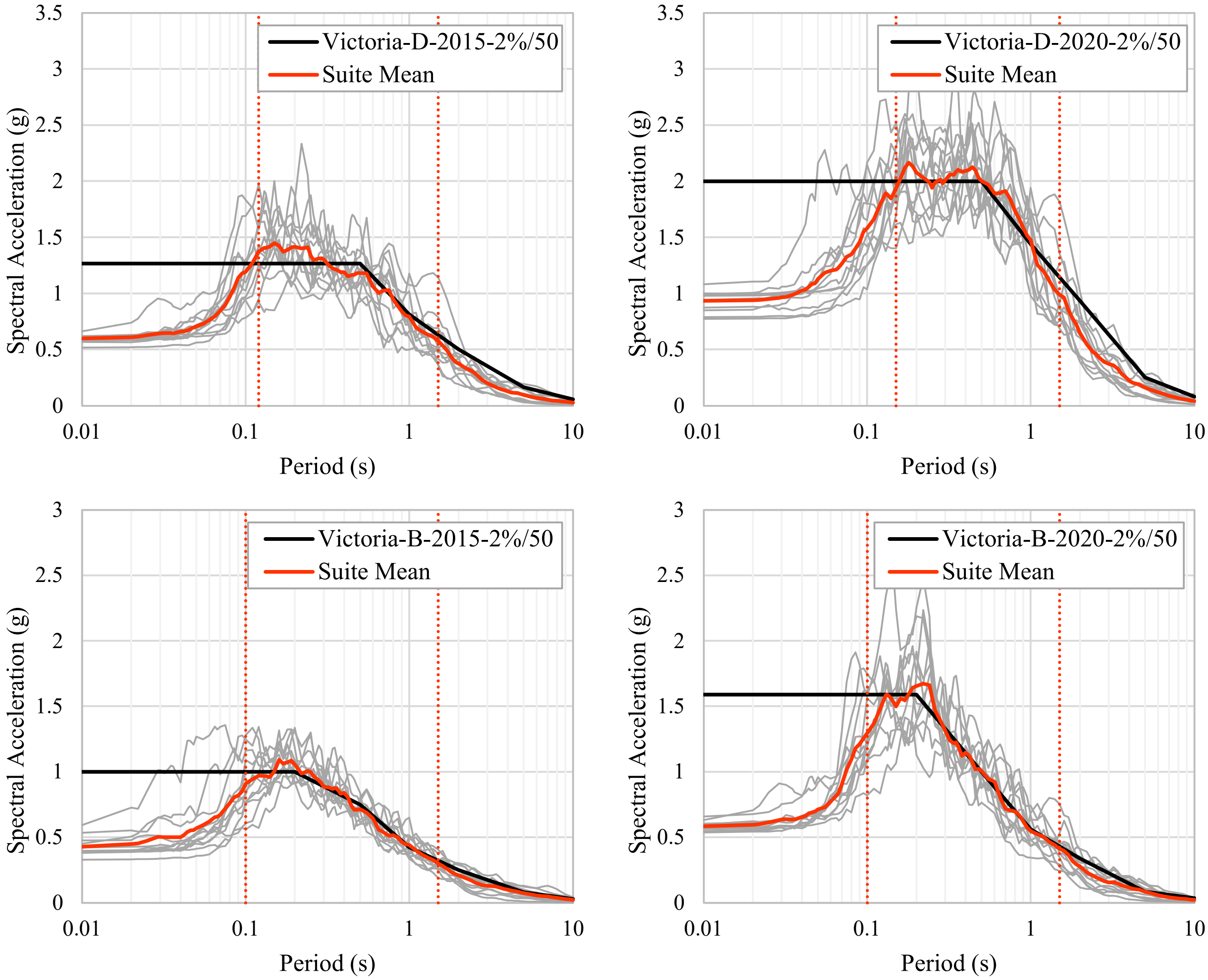

Figure 5. Selected ground motion records mean response spectra versus the design spectrum. The vertical lines mark the upper-bound and lower-bound periods for matching the suite mean to the target spectrum.

For case study 3, the suites of ground motion records selected for the 2%/50 NBC 2015 and NBC 2020 spectra were linearly scaled to match the 10%/50 spectra.

RESULTS AND DISCUSSION

To better help with interpreting the NTHA results within the context of this paper, two measures were selected to present the data, including:

- The ratio of demand to capacity of the component for shear and bending moment; and
- The ratio of the mean demand from the suite of motions matched to the NBC 2020 spectrum to the mean demand from the suite matched to the NBC 2015 spectrum.

The first ratio demonstrates how the demands from each ground motion record would compare against the capacity of the component. In addition to the demand from individual ground motions, the mean demand from the suite of motions is also provided for comparison.

The second ratio verifies by how much the seismic demand would increase when the bridge is evaluated for the NCB 2020 hazard values instead of NBC 2015 values.

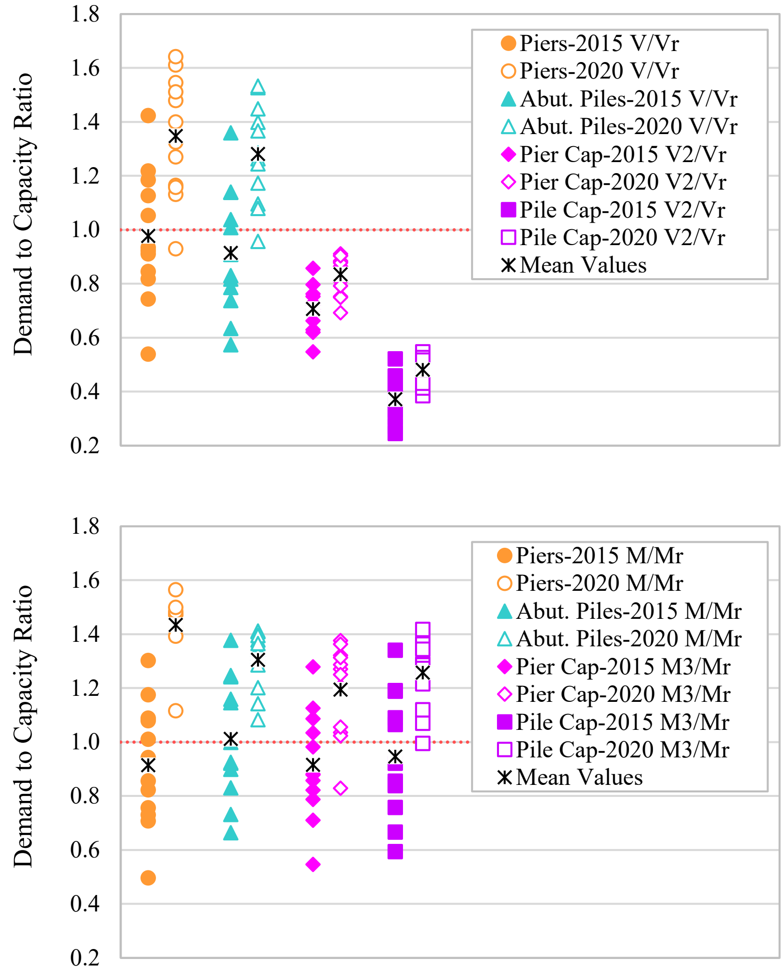

Figure 6. Demand to capacity ratios for the Victoria Site Class D 2%/50 suite of ground motions (case study 1)

Case Study 1

Figure 6 summarizes the demand-to-capacity ratios for case study 1. The ratio of the NBC 2020 to NBC 2015 demands are reported in Table 3. The results are presented for both shear, which is a brittle failure mode and flexure, which is a ductile failure mode. In the figure and the table, V is the resultant shear in the horizontal direction, V_2 and V_3 are the vertical and horizontal shear in the pier cap and abutment pile caps, and V_r is the factored shear resistance using expected material properties. Similarly, M is the resultant bending moment, M_3 and M_2 are the bending moments about the strong and weak axes of the pier cap and abutment pile caps, and M_r is the flexural resistance using expected material properties.

By examining the results, the following can be observed:

- The mean shear demands for the piers and abutment piles from the NBC 2015 suite of motions are close to the shear capacity of the components. Piers and abutment piles were designed for the 1.25 times of the elastic demands from RSA under the NBC 2015 design spectrum with no reserve capacity. It was expected that the mean shear demand for the NBC 2015 suite falls below the capacity line for each component due to incorporation of the 1.25 EFF. However, the results show that the shear demands from NTHA are about 20-25% larger than the shear demands from RSA. This increase is primarily due to modelling uncertainty related to better modelling of the nonlinear soil behavior at the boundaries and including geometric and other sources of nonlinearities, which are not effectively captured in an elastic analysis. Therefore, the incorporated 1.25 EFF in the design is essentially absorbed by the modelling uncertainties.

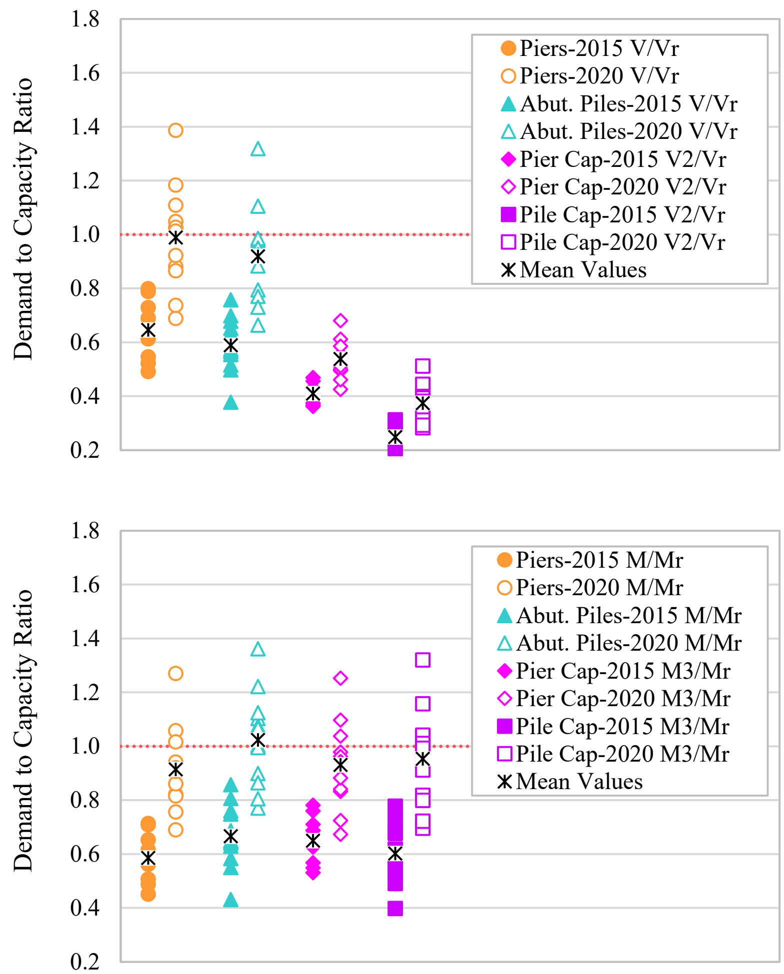

Figure 7. Demand to capacity ratios for the Victoria Site Class B 2%/50 suite of ground motions (*case study 2*)

- While the mean shear demand of the piers and abutment piles meet the capacity of the components, about 50% percent of the demands from individual ground motions exceeds the shear capacity. Since shear failures are brittle failures, one may argue that the local shear failure of the columns and abutment piles could lead to a life safety/probable replacement performance level for the bridge and collapse of the bridge. This is in conflict with the expected performance level of minimal damage intended to be achieved from an essentially elastic design. Therefore, a larger margin of safety would be required to address the record-to-record variability of the demands. This would entail using a larger EFF in the first place or allowing some reserve capacity when designing the marginally protected components.
- The mean shear demands for the pier and abutment piles for the NBC 2020 suite of motions are about 30-40% larger than the respective shear capacities of the components. Combined with the 25% reserve capacity incorporated through using the 1.25 EFF, this would result in the NTHA demands for the NBC 2020 suite of motions to be about 75% larger than the NBC 2015 elastic demands from RSA, for which the components were designed. This is while the ratio of the NTHA shear demands from the suite of NBC 2020 motions to NBC 2015 motions is about 1.4, indicating a 40% increase in the demands calculated from an inelastic method of analysis.
- The pier cap and pile caps have considerable reserve shear capacities, and as such, the inadequacy of the 1.25 EFF does not compromise their performance. This is because the design of these members was governed by other load cases and geometric constraints rather than seismic demands.
- The mean bending moment demand in the pier cap and pile caps from the NBC 2020 suite of motions are about 20-25% larger than the flexural capacities of the components. This would result in plastic hinges forming in the pier cap and pile caps, where it is not desired to have plastic hinges.

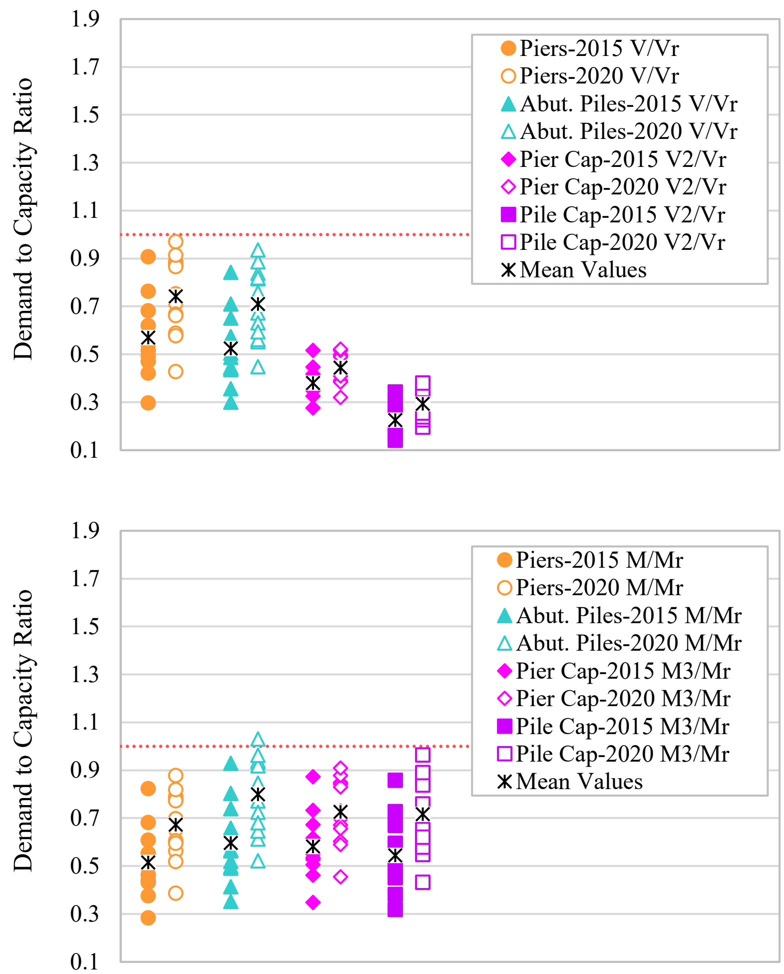

Figure 8. Demand to capacity ratios for the Victoria Site Class D 10%/50 suite of ground motions (case study 3)

- The ratios of the mean M_2 and V_3 demands from the NBC 2020 suite to the mean demands from the NBC 2015 suite for the pier cap and pile caps are considerably high, and in the order of 2.0 times larger or more. The increase in demands is larger for the longitudinal direction compared to the transverse direction of the bridge. This is primarily due to soil-structure interaction at the backfill soil spring in the longitudinal direction. The ratio of the mean deck displacement demand for NBC 2020 to NBC 2015 reported in Table 4 further shows a 4.5 time increase in the longitudinal displacement demands. This is due to the fact that under the NBC 2015 suite of motions, the backfill soil spring remain primarily within the elastic range, while when the bridge is subjected to the NBC 2020 suite of motions, the backfill soil spring yields, resulting in a much larger displacement demands in the longitudinal direction.
- The mean demand values were also calculated for subsets of crustal and subduction suite of motions. In all cases, no significant difference was observed between the mean demand values from the crustal and subduction suite of motions. This observation needs further investigation and should be verified for cases where the case study bridge has a longer fundamental period of vibration, etc.

Case Study 2

Figure 7 summarizes the demand-to-capacity ratios for case study 2 and the ratio of the NBC 2020 to NBC 2015 demands are reported in Table 3.

By examining the results, the following can be observed:

- Since the components were designed for Site Class D condition and larger demands, they have about 40% or more reserve capacity when subjected to the NBC 2015 suite of motions selected for Site Class B condition.

Table 3. Ratios of the mean force demands from the suite of ground motions scaled to the NBC 2020 design spectrum to the demands from the motions scaled to the NBC 2015 design spectrum for each analysis case.

		Resultant Force		Individual Force Component			
Case	**Component**	***V***	***M***	V_2	M_3	V_3	M_2
Victoria-D-2%/50 (Case study 1)	Pier Columns	1.38	1.57	-	-	-	-
	Abutment Piles	1.40	1.29	-	-	-	-
	Pier Cap	-	-	1.18	1.30	2.87	2.67
	Pile Cap	-	-	1.30	1.33	1.49	2.52
Victoria-B-2%/50 (Case study 2)	Pier Columns	1.53	1.56	-	-	-	-
	Abutment Piles	1.56	1.54	-	-	-	-
	Pier Cap	-	-	1.31	1.43	1.76	1.79
	Pile Cap	-	-	1.50	1.58	1.65	1.69
Victoria-D-10%/50 (Case study 3)	Pier Columns	1.30	1.31	-	-	-	-
	Abutment Piles	1.35	1.34	-	-	-	-
	Pier Cap	-	-	1.17	1.25	1.40	1.36
	Pile Cap	-	-	1.30	1.32	1.55	1.54

Table 4. Ratios of the mean displacement demands from the suite of ground motions scaled to the NBC 2020 design spectrum to the demands from the motions scaled to the NBC 2015 design spectrum for each analysis case.

Component	Case	D_x	D_y	D
Deck Centre	Victoria-D-2%/50	4.62	1.46	3.22
	Victoria-B-2%/50	1.67	1.59	1.64
	Victoria-D-10%/50	1.53	1.36	1.41

- The ratios of the mean demands from the NBC 2020 suite of motions to NBC 2015 suite of motions listed in Table 3, show minor changes from ratios calculated for Site Class D condition in case study 1. Other than some reduction in the ratios for M_2 and V_3 of the pier cap and pile caps, all other ratios are comparable with Site Class D condition. The lower ratios for M_2 and V_3 are due to lower soil-structure interaction in the longitudinal direction at the backfill soil springs of the Site Class B model. This observation appears to be inconsistent with the NBC 2020 to NBC 2015 spectral acceleration ratios for Site Class B and Site Class D shown in Figure 1, since at the fundamental period of the bridge in the longitudinal direction (0.5-0.7 s) the ratio is larger for site Class D. One explanation for this observation could be that the fundamental period of the bridge in the transverse direction is 0.29 s and at that period, the NBC 2020 to NBC 2015 spectral acceleration ratio for Site Class B is actually higher than Site Class D. Since the response of the bridge is a combination of the longitudinal and transverse responses, the ratios of the demands from NTHA do not vary considerably for Site Class B and Site Class D conditions. To clearly show the effect of site soil properties on ratio of the NBC 2020 to NBC 2015 mean demands, further studies are needed to isolate the effect.

Case Study 3

Figure 9 summarizes the demand-to-capacity ratios for case study and the ratio of the NBC 2020 to NBC 2015 demands are reported in Table 3.

By examining the results, the following can be observed:

- Since the components were designed for the 2%/50 demands, they have about 25% or more reserve capacity when subjected to the suite of motions scaled for the 10%/50 hazard level.
- The ratios of the mean demands from the NBC 2020 suite of motions to NBC 2015 suite of motions listed in Table 3, show minor reductions from those calculated for the 2%/50 hazard level in case study 1. This is somewhat expected as the NBC 2020 to NBC 2015 spectral acceleration ratios for Site Class D at 2%/50 is only slightly larger than 10%/50 at the fundamental periods of the bridge (see Figure 1). Reductions in ratios for M_2 and V_3 of the pier cap and pile caps are larger due to lower soil-structure interaction in the longitudinal direction at the backfill soil springs at the 10%/50 hazard level compared to the 2%/50 level.

CONCLUSIONS

An in-depth critical review of the Elastic Force Factor in CSA S6 was presented. The current inclusion of the EFF as part of the capacity-design provisions in CSA S6:19 is inherently in conflict with the philosophy of capacity-design. To resolve this issue, designing with the EFF should be treated as a proxy to capacity-design, and the components designed using the proxy design approach would be more appropriately referred to as marginally protected. It was verified further that the current 1.25 EFF in CSA S6:19 could be insufficient to safeguard against brittle failure modes. As a demand factor, it was discussed that the EFF is a function of aleatory and epistemic uncertainties in demand and capacity. An adequate EFF value can only be determined if all sources of uncertainty have been taken into account and the EFF is calibrated using a probabilistic approach to a target reliability. Further research would be required to calibrate the EFF.

The effect of site soil classification, ground motion return period and type of earthquake (seismic hazard model uncertainty), modelling uncertainty, and record-to-record variability on the EFF was showcased and discussed through three case studies of a model RC bridge in Victoria, BC. It was demonstrated that the 25% safety margin provided by the 1.25 EFF could be absorbed by just the modelling uncertainty, rendering the EFF ineffective to safeguard against brittle failure modes. It was further discussed that in order to avoid forming plastic hinges in undesirable locations (such as pier caps or pile caps) a hierarchy of strength between marginally protected components and ductile components needs to be established. This can be achieved by factoring the flexural demands (force demands for ductile failure modes) in marginally protected components by the EFF, in addition to factoring the shear demands (force demands for brittle failure modes) by the EFF. It was also shown that allowing reasonable reserve capacities for marginally protected components can lead to a more reliable safety margin against undesirable failure modes in light of the inadequacy of the 1.25 EFF. The observations for the effect of site soil properties on the EFF, indicated that further research is required to more effectively isolate the effect of site soil class and shear wave velocity on the EFF. The observed effect of the type of earthquake and ground motion return period on the EFF was minimal. However, this would also require a more thorough investigation on a larger number of case study bridges.

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