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RESULTS OF SEVERAL EASTERN CANADA SITE RESPONSE ANALYSES

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ABSTRACT
Seismic response analyses at sites in eastern Canada is becoming more common than in the past with the typical approach being to use an equivalent linear site response model. The authors conducted such site response analyses for several building and bridge projects in eastern Canada over the past years under a variety of design earthquake hazard levels, ground conditions and design objectives. The paper discusses the details of several case histories, the approaches used and observations on the outcomes achieved.

Of interest will be the observation that in these cases the seismic ground response is lower than would have been achieved using code-specified spectral values or cyclic-stress ratios obtained using simpler methods of analyses. The results obtained represent valuable examples of the value of these types of analyses when properly done and would suggest that their more routine usage is warranted from a cost-benefit perspective.

RÉSUMÉ
Les analyses de réponse sismique sur les sites de l'est du Canada sont de plus en plus courantes, l'approche typique étant d'utiliser un modèle linéaire équivalent de réponse au site. Les auteurs ont effectué de telles analyses d'intervention du site pour plusieurs projets de construction et de ponts et de bâtiments dans l'est du Canada au cours des dernières années comprenant une variété de niveaux d'aléas sismique, de conditions au sol et d'objectifs de conception. Le document traite des détails de plusieurs histoires de cas, des approches utilisées et des observations sur les résultats obtenus.

Il sera intéressant de noter que, dans ces cas, la réponse sismique au sol est inférieure à ce qui aurait été atteint à l'aide de valeurs spectrales spécifiées par code ou de profil de contraintes sismiques en cisaillement obtenus à l'aide de méthodes d'analyse plus simples. Les résultats obtenus représentent de précieux exemples de la valeur de ces types d'analyses lorsqu'ils sont bien effectués et suggèrent que leur utilisation plus courante est justifiée du point de vue coûts-avantages.

1 INTRODUCTION

Simplified methods are typically used to calculate both the seismic design response spectrum and the cyclic stress ratio (CSR) profile and for geotechnical seismic design and analysis of buildings and bridges in accordance with the Ontario Building Code (OBC) (Ontario 2012) or National Building Code of Canada (NBCC) (National Research Council 2010, 2015) for buildings, and with the Canadian Highway Bridge Design Code (CHBDC) (CSA group, 2014a,b) for bridges, which are consistent in their proposed methodologies for analysis of both CSR and seismic design spectra. Given the span of time over which this work was completed, the applicable version of the code varies.

The CSR of a soil layer is used for the assessment of the potential for liquefaction or cyclic mobility and represents the seismic demand on a given ground layer during a seismic event. The simplified method often used to calculate a CSR profile outlined in the CHBDC follows the recommendations of Idriss and Boulanger (2008), and is estimated from the peak ground acceleration (PGA) of the site, the effective/total stresses within the soil layer, and a stress reduction factor, rd. The CSR represents a normalization of the shear stresses induced by the earthquake within the ground. The CSR values are then compared to the cyclic resistance ratio (CRR) of the ground, which represents a normalization of the seismic shear strength of the ground and is typically approximated from results of in-situ testing. Liquefaction is predicted to occur when the CSR of a soil layer is greater than the CRR of the same layer.

A seismic design spectrum represents the demand site of the earthquake loads that need to be resisted by structures. The simplified method for generating a seismic design spectrum is codified in the aforementioned codes and consists of 3 steps: 1) obtain reference spectral values developed from seismological and attenuation models; 2) assess the type of ground conditions that may affect the reference spectral values by selecting an appropriate Site Class; and 3) adjusting the reference spectral values for selected Site Class. The Seismic Site Class is selected based on the weighted average of measured soil properties, either SPT N_{60}, shear strength s_u, or shear wave velocity V_s, of the 30 metres of ground below the foundation depth of the structure and is given a designation between A and E. For all the sites presented herein, site-specific measurements of Vs were used.

The National Research Council (NRC) Earthquakes Canada website provides site-specific reference spectral values for Site Class C (https://www.earthquakes canada.nrcan.gc.ca/). For Site Classes A, B, D, and E, the seismic spectral design values for the structure are calculated based on linear scaling factors for the provided reference Site Class C values which are provided on the NRC website for the PGA, as well as spectral acceleration periods of 0.2, 0.5, 1.0, and 2.0 seconds for the OBC and includes values and factors for spectral acceleration periods of 5.0 and 10.0 seconds for the NBCC and CHBDC.

A site is designated as Seismic Site Class F if it meets specific criteria as outlined in the applicable code (e.g. liquefiable soils present below the structure foundation level, more than 30 metres of soft to firm clays present, etc.).

The paper outlines the results of the site specific ground response analyses carried out for nine sites in eastern Canada which were designated as Site Class F due to potential for liquefaction or the presence of deep silty clay deposits in accordance with the CHBDC for bridge sites and OBC or NBCC for building sites. All ground response analyses were carried out using the software Shake2000 (Version 99.99.93, released June 2015, part of the Professional Suite of ground response software by GeoMotions, LLC).

2 SITE DESCRIPTIONS

A summary table of the site structure, applicable code, and firm ground spectra site class is presented in Table 1.

Table 1. Site Summaries

Site #	Structure	Code	Firm Ground Site Class	Design Earthquake Return Period (year)
1	Bridge	CHBDC-S-14	A	2,475, 975, 475
2	Building	NBCC 2015	A	2,475
3	Bridge	CHBDC-S-14	B	2,475
4	Building	OBC 2012	B	2,475
5	Building	OBC 2012	A	2,475
6	Bridge	CHBDC-S-14	B	2,475
7	Bridge	CHBDC-S-14	B	2,475
8	Building	NBCC 2010	C	2,475
9	Bridge	CHBDC-S-14	B	2,475, 975

For conciseness, only the results of the 2475-year earthquake return design period analyses will be discussed herein.

2.1 Site 1

The analyses for Site 1 were carried out for a set of twin fifteen-span bridges approximately 385 metres in length in the southern region of Ottawa, Ontario. Shear wave velocity data was obtained from the results of vertical seismic profile (VSP) testing carried out at the site. Two stratigraphic profiles were considered for the analyses to capture the variability of conditions across the site. The ground conditions generally consisted of deposits of silty clay overlying glacial till and bedrock, where the weighted Vs of the 10.5 to 11.5 metres of overburden soils was between 376 and 550 m/s and the characteristic Vs of the underlying bedrock was 1,900 m/s.

2.2 Site 2

The analyses for Site 2 were carried out for a multi-building campus about 500 metres by 150 metres in plan and located in Quebec City, Quebec. Shear wave velocity data was obtained from the results of multi-channel analysis of surface waves (MASW) testing carried out at the site. One stratigraphic profile was considered for the analyses, generally consisting of surficial fill and interbedded deposits of silt and sand overlying bedrock, where the weighted Vs of the 65 metres of overburden soils was 315 m/s and the characteristic Vs of the underlying bedrock was 1,500 m/s.

2.3 Site 3

The analyses for Site 3 were carried out for a set of twin, three-span and four-span bridges approximately 79 metres long and located in the eastern region of Ottawa, Ontario. The shear wave velocity data was obtained from the results of MASW testing carried out at the site. One stratigraphic profile was considered for the analyses, generally consisting of silty sand overlying bedrock, where the weighted Vs of the 12.5 metres of overburden soils was 478 m/s and the characteristic Vs of the underlying bedrock was 838 m/s.

2.4 Site 4

The analyses for Site 4 were carried out for a 50 metres by 60 metres, 8 storey structure located in the central region of Ottawa, Ontario. The shear wave velocity data was obtained from the results of MASW testing carried out at the site. One stratigraphic profile was considered for the analyses, generally consisting of surficial granular fill and silty clay overlying interbedded sands and silts, glacial till, and bedrock, where the weighted Vs of the 31.5 metres of overburden soils was 365 m/s and the characteristic Vs of the underlying bedrock was 1,200 m/s.

2.5 Site 5

The analyses for Site 5 were carried out for a 40 metres by 90 metres, 9 storey structure located in the eastern region of Ottawa, Ontario. The shear wave velocity data was obtained from the results of VSP testing carried out at the site. Two stratigraphic profiles were considered for the analysis, generally consisting of surficial fill and silty clay overlying glacial till and bedrock, where the weighted Vs of the 16 to 25 metres of overburden soils was between 311 and 418 m/s and the characteristic Vs of the underlying bedrock was 1,700 m/s.

2.6 Site 6

The analyses for Site 6 were carried out for a four-span bridge approximately 75 metres long and located about 75 kilometres southeast of Ottawa, Ontario. The shear wave velocity data was obtained from the results of seismic CPT testing carried out at the site. One stratigraphic profile was considered for the analyses, generally consisting of surficial fill overlying glacial till and bedrock, where the weighted Vs of the 25 metres of overburden soils was 488 m/s and the characteristic Vs of the underlying bedrock was 760 m/s.

2.7 Site 7

The analysis for Site 7 was carried out for a single-span bridge approximately 48 metres long and located near Peterborough County, Ontario. The shear wave velocity data was obtained from the results of seismic CPT testing carried out at the site. One stratigraphic profile was considered for the analysis, generally consisting of surficial fill overlying interbedded sands and silts, including organic sands and silts, as well as cobbles and boulders followed by bedrock, where the weighted Vs of the 19 metres of overburden soils was 223 m/s and the characteristic Vs of the underlying bedrock was 760 m/s.

2.8 Site 8

The analysis for Site 9 was carried out for a multi-structure campus located northwest of Montreal, Quebec. The shear wave velocity data was obtained from the results of VSP testing carried out at the site. One stratigraphic profile was considered for the analysis, generally consisting of surficial fill overlying silty clay and glacial till at depth, where the weighted Vs of the 31.5 metres of overburden soils was 160 m/s and the characteristic Vs of the underlying firm ground glacial till was 500 m/s.

2.9 Site 9

The analysis for Site 9 was carried out for a set of twin, three-span bridges located near Lancaster, Ontario. The shear wave velocity data was obtained from the results of VSP testing carried out at the site. Two stratigraphic profiles were considered for the analysis, generally consisting of surficial fill and silty clay overlying glacial till and bedrock, where the weighted Vs of the 9.3 to 15.6 metres of overburden soils was between 137 and 506 m/s and the characteristic Vs of the underlying bedrock was 1,000 m/s.

3 ANALYSIS INPUT AND METHODS

3.1 Earthquake Input Time History Selection

The process to select appropriate input ground motion time histories for each site is consistent with procedures outlined in CHBDC or the NBCC.

An input target spectrum for the firm ground was developed for each return period using the NRC reference spectral values and the codified process to adjust these as needed for their characteristic Site Class.

Seismic hazard deaggregations, for each return period, were then used to identify the primary seismic events contributing to the hazard, and their associated earthquake magnitude and hypocentral distance. The hazard deaggregations were sourced from the National Research Council (NRC) (2019).

A suite of representative seed input time histories that aligned with the deaggregation data and the periods of interest for the analyses were then selected for each design earthquake return period. Selection criteria generally included earthquake magnitude, source-to-site distance, style of faulting, rupture directivity, and site ground condition; however, in general, and particularly in eastern Canada, there are often insufficient recordings in the earthquake strong motion databases to meet the desired selection criteria (Assatourians and Atkinson 2019). Thus, to acquire enough candidate empirical acceleration time-history recordings, some of these conditions (e.g., style-of-faulting and site soil condition) are often relaxed. The time histories used for each analysis were selected from either the UC Berkeley Pacific Earthquake Engineering Research (PEER) Center NGA-West2 database (PEER 2020), consisting of worldwide recorded ground motions of shallow crustal earthquakes, or from the Engineering Seismology Toolbox (EST) database, consisting of synthetic ground motion records developed for analyses in eastern Canada (EST 2020).

The time histories were then scaled to match the firm ground target spectra to represent the site-specific design firm-ground accelerations, for use in the site-specific ground response analyses.

The literature describes two suitable approaches to scaling input time histories to match a target spectrum: linear scaling and spectral matching.

Linear scaling involves scaling the ordinates of the time history record to achieve the best fit to the target response spectrum over the period range of interest. Linear scaling provides input time histories that are more representative of the original records of ground shaking (i.e. less modification), but can be difficult to match the target spectrum over a large period range of interest.

Spectral matching involves changing the frequency and phase contents of the record to match the target spectrum. Spectral matching allows for development of input records that provide a closer match to the target spectrum over a broad range of periods but involves more modification of the original records since no real earthquake spectrum will match the entire target spectrum.

A summary of the time history selection and scaling methodology for each site is presented in Table 2.

Table 2. Summary of Time History Selection and Scaling

Site #	# of PEER Time Histories	# of EST Time Histories	Scaling Method
1	4	7	Linear
2	6	7	Linear
3	10	1	Linear
4	10	8	Linear
5	10	2	Linear
6	12	0	Spectral
7	8	3	Linear
8	8	0	Linear
9	10	5	Spectral – PEER Linear – EST

Figures 1, 2, and 3 below show the input (firm ground) spectra for Sites 2, 6, and 9 respectively:

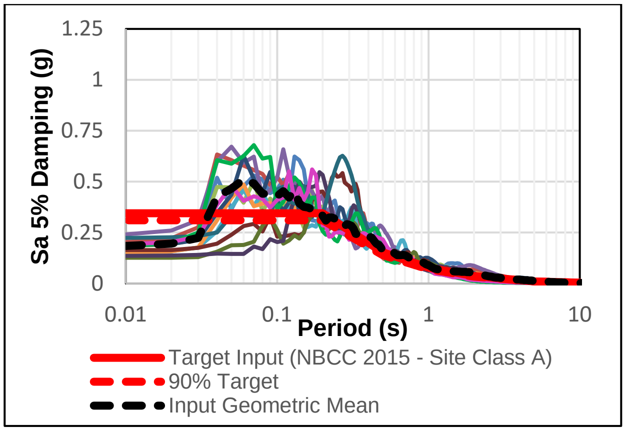

Figure 1. Site 2 Firm Ground Input Spectra

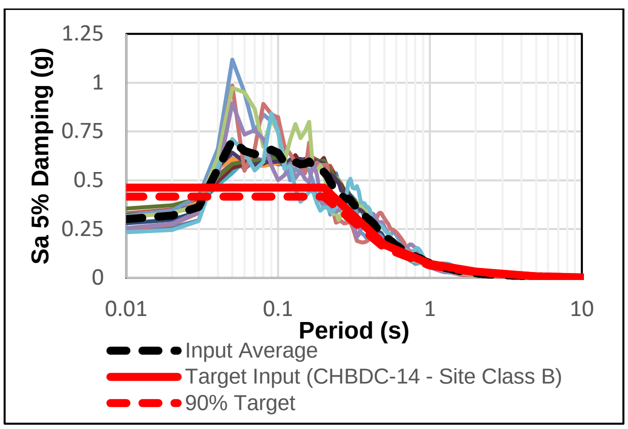

Figure 2. Site 6 Firm Ground Input Spectra

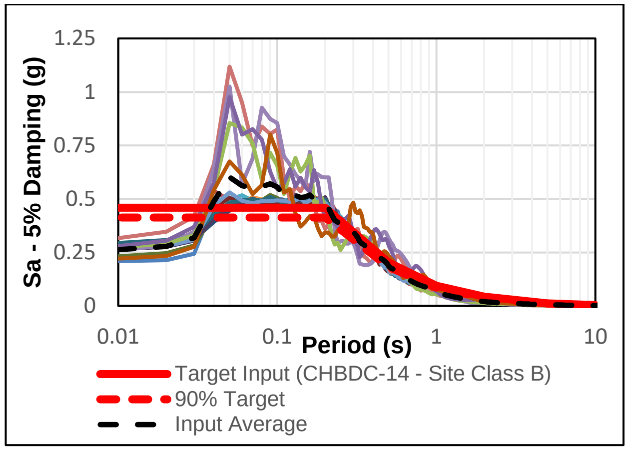

Figure 3. Site 9 Firm Ground Input Spectra

3.2 Soil Profiles and Material Input

One or two representative soil profiles were developed for each site. The soil profiles were input into the analyses in layers of discrete thickness of one metre or less. A damping value of 5% was considered for soil, and a value of 2% was considered for bedrock. The shear wave velocity assigned to each layer in the model was based on the shear wave velocity testing carried out at each site.

The soil unit weights were selected based on the results of soil testing and on experience with nearby soils in the areas of the sites.

The modulus and damping curves used for each stratigraphic material are noted by material ID in the site tables, respectively, and summarized in Table 3 summaries.

The following Tables 3 to 14 summarize the soil stratigraphy input for each site profile analyzed.

Table 3. Stratigraphy Profile - Site 1, Profile 1:

Soil Unit	γ (kN/m^3)	Depth (m)	Vs (m/s)	Material ID
Silty Clay	16	0 – 5.0	116-203	5
Silty Clay /Clayey Silt	18	5.0–6.0	291–680	5
Glacial Till	20	6.0–11.5	1070–1080	2
Bedrock	23	> 11.5	1900	7

Table 4. Stratigraphy Profile – Site 1, Profile 2

Soil Unit	γ (kN/m^3)	Depth (m)	Vs (m/s)	Material ID
Silty Clay	16	0 – 7.5	110 – 200	5
Silty Clay /Clayey Silt	18	7.5 – 8.3	320 – 510	5
Glacial Till	20	8.3 – 10.5	610 – 990	2
Bedrock	23	> 10.5	1900	7

Table 5. Stratigraphy Profile - Site 2

Soil Unit	γ (kN/m^3)	Depth (m)	Vs (m/s)	Material ID
Fill	20	0–0.5	189	2

Silt	18	0.5–5.5	189 – 230	6
Sand	19	5.5–35.5	226 – 395	2
Silt	16.5	35.5–51.0	301 – 337	6
Cohesion-less Soil	19	51.0–65.0	321 – 372	1,3
Bedrock	26	>65.0	1500	7

Table 6. Stratigraphy Profile – Site 3

Soil Unit	γ (kN/m^3)	Depth (m)	Vs (m/s)	Material ID
Silty Sand	20	0 – 12.5	320 – 670	2
Bedrock	23	>12.5	840	8

Table 7. Stratigraphy Profile - Site 4

Soil Unit	γ (kN/m^3)	Depth (m)	Vs (m/s)	Material ID
Fill	20	0 – 2.5	143 - 165	2
Silty Clay	17.5	2.5 – 12.5	155 - 239	6
Silt, Silty Sand and Sandy Silt	20	12.5 – 24.5	239 - 396	2
Glacial Till	21	24.5 – 31.0	396 - 645	2
Bedrock	26	>31.0	1,200	8

Table 8. Stratigraphy Profile - Site 5, Profile 1

Soil Unit	γ (kN/m^3)	Depth (m)	Vs (m/s)	Material ID
Fill – Sand	18	0 – 1	260	2
Silty Clay – Weathered Crust	17	1 – 4	180 – 220	5
Silty Clay	16	4 – 9	175-315	5
Glacial Till	22	9–16	405 – 630	6
Bedrock	23	>16	1700	8

Table 9. Stratigraphy Profile - Site 5 Profile 2

Soil Unit	γ (kN/m^3)	Depth (m)	Vs (m/s)	Material ID
Fill – Sand	18	0 – 1	260	2
Silty Clay – Weathered Crust	17	1 – 4	180 – 220	5
Silty Clay	16	4 – 9	175-315	5
Glacial Till	22	9 – 24	405 – 630	6
Bedrock	23	>24	1700	8

Table 10. Stratigraphy Profile - Site 6

Soil Unit	γ (kN/m^3)	Depth (m)	Vs (m/s)	Material ID
Fill	21.5	0 – 3.0	350	1,3
Till	21	3.0 – 24.5	400 – 559	1,3
Sand	19	24.5–25.0	500	1,3
Bedrock	26	>25.0	760	8

Table 11. Stratigraphy Profile - Site 7

Soil Unit	γ (kN/m^3)	Depth (m)	Vs (m/s)	Material ID
Fill	21	0–1.0	84	1,3
Silty Sand	20	1.0–2.0	84	1,3
Sand, some silt	18	2.0–3.5	84 – 119	1,3
Sand, trace silt	19	3.5–7.5	130 – 144	1,3
Organic Sand	16	7.5–13.0	117 – 134	1,3
Organic Silt	16	13.0–14.5	120	1,3
Cobbles/ Boulders	21	14.5–19.0	560	4
Bedrock	25	>19.0	760	8

Table 12. Stratigraphy Profile - Site 8

Soil Unit	γ (kN/m^3)	Depth (m)	Vs (m/s)	Material ID
Granular Fill	20	0 - 2	183	2
Silty Clay – Weathered Crust	17	2 - 5	152 - 160	5
Silty Clay	16	5 - 36	107 - 250	5
Glacial Till	22	36+	500	2

Table 13. Stratigraphy Profile - Site 9, Profile 1

Soil Unit	γ (kN/m^3)	Depth (m)	Vs (m/s)	Material ID
Granular Fill	18	0 – 2.3	150-190	2
Silty Clay	16	2.3-3.3	205-220	5
Glacial Till	22	3.3-15.6	250-720	5
Bedrock	26	>15.6	1,000	8

Table 14. Stratigraphy Profile - Site 9, Profile 2

Soil Unit	γ (kN/m^3)	Depth (m)	Vs (m/s)	Material ID
Sand and Gravel	20	0-4.4	150	2
Silty Sand	18	4.4-5.3	115-135	2
Silty Clay	16	5.3-5.9	100	5
Glacial Till	22	5.9-9.3	135-200	5
Bedrock	26	>9.3	1,000	8

The modulus and damping versus shear strain curves were applied to the soil profile in the models as noted above and were generated by the Shake2000 program. A summary of the Material IDs is presented in Table 15.

Table 15. Modulus and Damping Curve Material IDs

Material ID	Modulus and Damping Curves
1	Seed and Idriss (1970) Lower curves for shear modulus and damping
2	Seed and Idriss (1970) Average curves for shear modulus and damping

3	Seed and Idriss (1970) Upper curves for shear modulus and damping
4	Seed et. al. (1986) Average curves for shear modulus and damping
5	Vucetic and Dobry (1991) curves for shear modulus and damping, I_p = 30%
6	Vucetic and Dobry (1991) curves for shear modulus and damping, I_p = 15%
7	Schnabel (1973) curves for shear modulus and damping
8	EPRI (1993) curves for shear modulus and damping

Where required for analysis, the small-strain shear modulus (G_{max}) for the site soils encountered within the depth of investigation were estimated using the site-specific shear wave velocity (V_s) measurements obtained from the results of the shear wave velocity testing at the sites.

3.3 Modelling Input

The one-dimensional soil columns and soil parameters described above were used for the ground response analyses. For all soil columns, the firm ground input motions were applied as within motions at the base of the soil column in the model (top of the bedrock or firm ground) to account for the overburden effects.

4 DISCUSSION AND RESULTS

The analyses output consisted of depth profiles for PGA, CSR, % strain, and shear modulus as well as 5% damped response spectra at the ground surface, foundation depth, and firm-ground depth. Only a review of the CSR and response spectra results are presented herein.

4.1 CSR Profile results

CSR profiles were calculated for all sites prior to completing the site-specific ground response analyses at all sites, with the exception of Site 2 and Site 8, in order to assess for the potential for liquefaction at the sites.

Figure 4 presents a summary of the percent CSR change with depth, relative to the simplified method, for each Site where CSR profiles were developed.

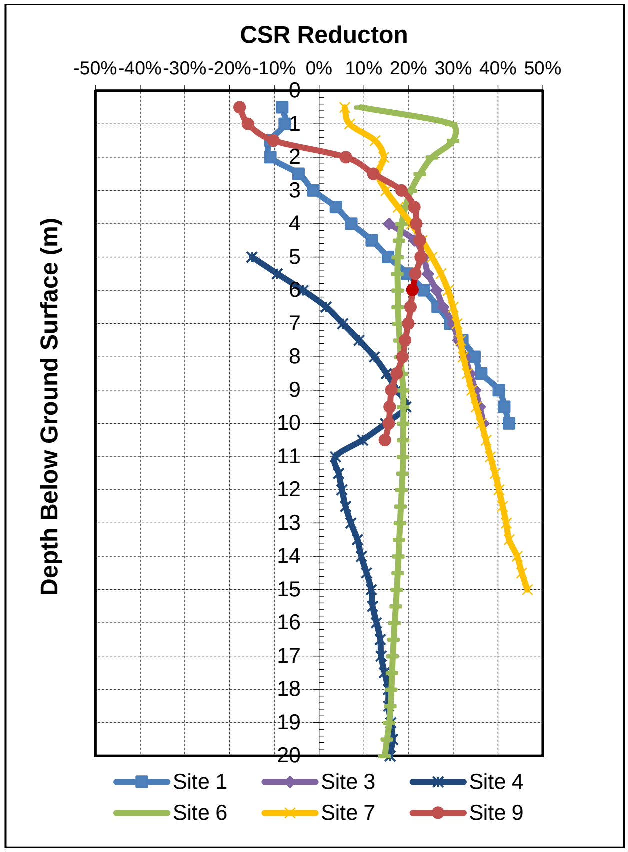

Figure 4. Percent CSR Relative Change with Depth

A summary of the of the integrated average CSR profile reduction over the depth of the profile for each site where simplified calculations were completed is presented in Figure 5.

As outlined above, the CSR profiles reduced between 10 and 31 percent when using the CSR profile results of the site-specific analysis in comparison to the CSR profile results of the simplified calculation method. The overall improvement of the CSR profile at each site accordingly reduced or eliminated the thickness of soil column where potential for liquefaction was identified.

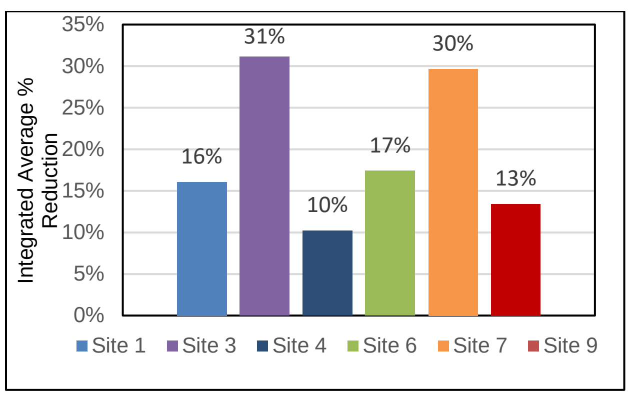

Figure 5. Summary of Integrated Average CSR Reduction

4.2 Design Spectra Results

Site-specific design spectra were generated for Sites 2, 4, 5, 6, 8, and 9. Site-specific design spectra were not generated for Sites 1, 3, and 7, where the analyses were carried out in support of generating a site-specific CSR profile to refine the liquefaction analysis for the site.

The following Figures 6 to 11 show comparisons between the code-specified design spectrum for each site (i.e. the design spectrum that would have been considered if a site-specific analysis had not been required) and, the geometric mean of the results of the site-specific response analysis for all sites. The firm ground input spectra are also provided for comparison of input and output results.

A summary of the percent change in code specified spectral values of the geometric mean value at periods of 0.1s, 0.2s, 0.5s, 1.0s, 2.0s, 5.0s and 10.0s is also presented in Table 16 where the approximate site natural period (4H/Vs) of the overburden soils is also presented.

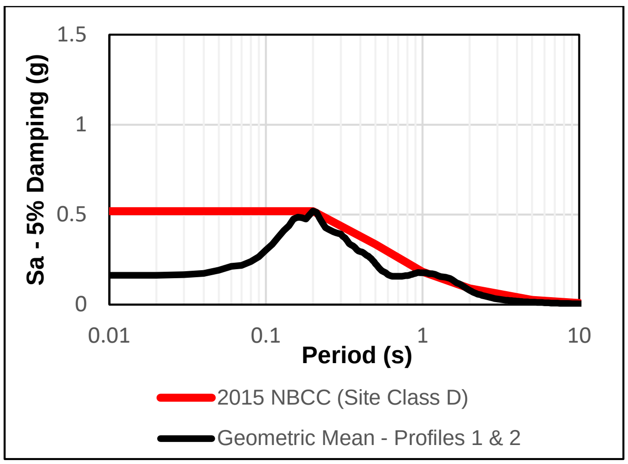

Figure 6. Site 2 Design Spectra Comparison

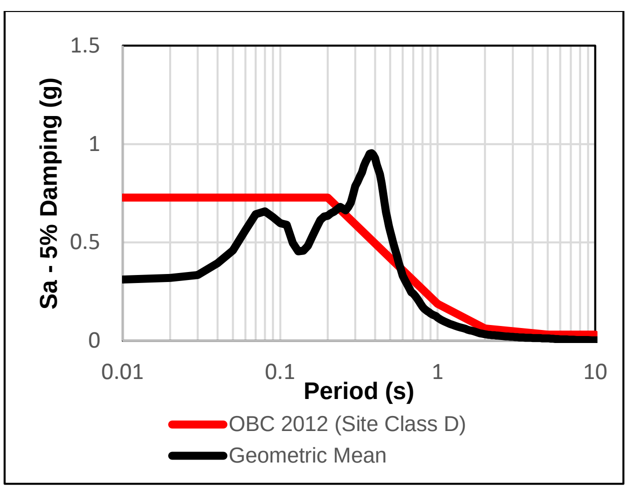

Figure 7. Site 4 Design Spectrum Comparison

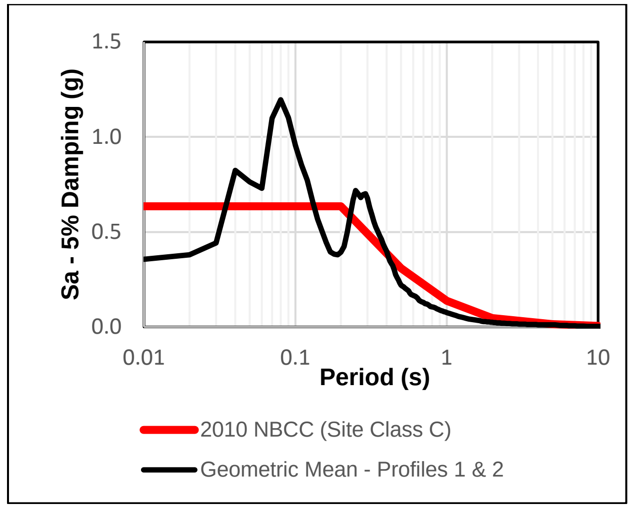

Figure 8. Site 5 Design Spectrum Comparison

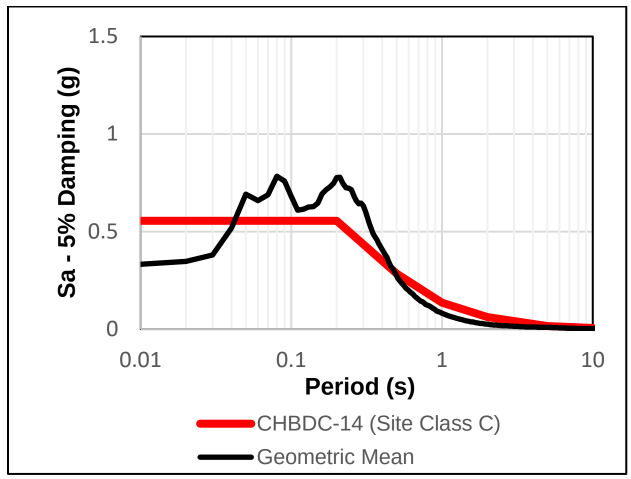

Figure 9. Site 6 Design Spectrum Comparison

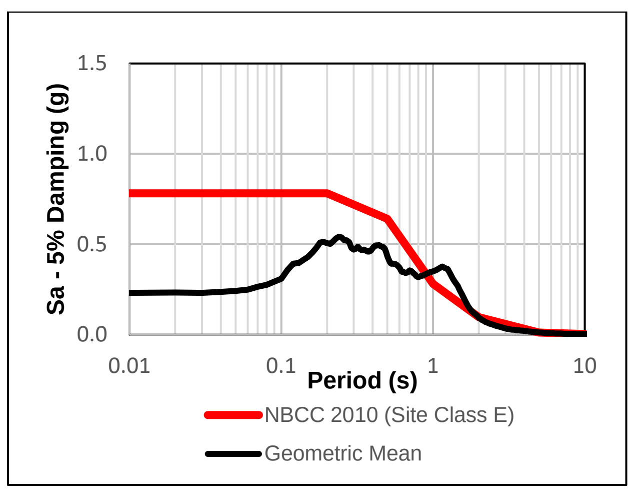

Figure 10. Site 8 Design Spectrum Comparison

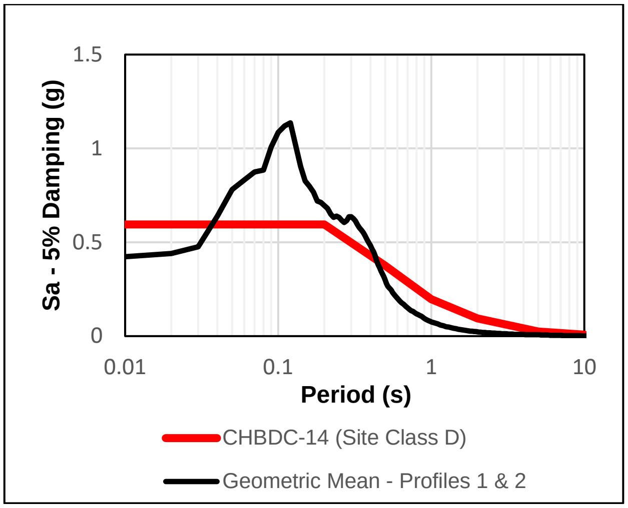

Figure 11. Site 9 Design Spectrum Comparison

Table 16. Summary of Spectral Acceleration Percent Change (positive values are a reduction)

Period (s)	Site 2	Site 4	Site 5	Site 6	Site 8	Site 9
0.1	59%	18%	-50%	-22%	60%	-82%
0.2	74%	13%	38%	-38%	35%	-17%
0.5	45%	-32%	29%	5%	32%	20%
1	9%	37%	45%	40%	-24%	61%
2	15%	47%	48%	61%	3%	76%
5	49%	62%	27%	53%	-6%	72%
10	56%	92%	67%	72%	26%	80%
Natural Site Period (s)	0.8	0.3	0.2	0.2	0.8	0.1 - 0.3

It may be noted that in some cases, the amplification of short period ground motions is much higher than expected for some earthquake records and ground conditions.

The results also indicate a general trend in the site-specific ground response spectra wherein the site-specific ground response analysis typically provides more favourable (i.e. lower) than the code-specific design spectra at periods outside the approximate fundamental period of the soil column at the site. Where the period of interest of the proposed structure is outside of the fundamental period of the soil column, the design spectrum improvement may be more pronounced.

5 CONCLUSIONS

A review of nine site-specific response analyses indicates a relatively consistent reduction in simplified CSR profiles under a variety of design earthquake hazard levels, ground conditions and design objectives. In completing these analyses, it has been observed that the results of the one-dimensional site-specific ground response analyses are generally more favourable than those provided using simplified methodologies.

When comparing the results of the CSR profiles within the soil columns to the CSR profiles calculated using the simplified method outlined in Idriss and Boulanger (2008), the site-specific response analyses typically provide a reduction in the CSR profile, with results showing improvements of between 10 and 31% over the depth of the profiles.

When comparing the results of these site specific ground response analysis spectrum to the code-defined site spectra, the results of the site-specific analyses typically provide a reduction in the response spectrum of between (i.e. the resulting spectrum generally has lower spectral values), and is more pronounced for sites where the anticipated fundamental period of the proposed structure differs from the natural period of the site soil column. Thus, for projects where the period of interest differs materially from the natural period of the overburden soils, the results typically show reductions in spectral values. Conversely, the effects of the site-specific analyses do show amplifications at times greater than code-specified values typically in around the natural period of the overburden soils.

The results obtained represent valuable examples of the value of these types of analyses when completed, and would suggest that their more routine usage is warranted from a cost-benefit perspective for sites which may benefit from a potential reduction in soil thickness where liquefaction potential is identified, or where seismic design of the structure would benefit from an overall reduction in the proposed seismic design spectrum.

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