Seismic design of CSP culverts in Eastern Canada

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ABSTRACT
In Canada and around the world there is a growing trend to use large span soil-steel arch structures as a substitute for the more conventional types of bridges and rigid culverts. In this study, full dynamic finite element modeling has been carried out to investigate the seismic performance of large span steel corrugated plate culverts in Eastern Canada and in different site conditions, whereas, the seismic loading has a significant effect on the deformations and the internal forces (moment and thrust forces). In addition, assessment of the seismic design according to the Canadian Highway Bridge Design Code (CHBDC) for CSP culverts is examined by comparing the results of the CHBDC with the results of the conducted finite element analyses. The study indicates that the CHBDC's equations can be used safely in the regions of Eastern Canada with low seismicity.

RÉSUMÉ
Au Canada et ailleurs dans le monde, on observe une tendance croissante à utiliser des structures d'arches en acier et en acier de grande envergure pour remplacer les types de ponts et les ponceaux rigides les plus conventionnels. Dans cette étude, une modélisation complète par éléments finis dynamiques a été réalisée pour étudier la performance sismique des ponceaux en tôle ondulée de grande portée dans l'est du Canada et dans différentes conditions de site, tandis que le chargement sismique a un effet significatif sur les déformations (forces de moment et de poussée). De plus, on examine l'évaluation de la conception sismique selon le Code canadien de conception des ponts routiers (CHBDC) pour les ponceaux CSP en comparant les résultats du CHBDC avec les résultats des analyses par éléments finis effectuées. L'étude indique que les équations du CHBDC peuvent être utilisées en toute sécurité dans les régions de l'est du Canada où la sismicité est faible.

1 INTRODUCTION

It is well known that over the past ten decades, large span corrugated steel plate (CSP) culverts are widely used in multipurpose underground utilities, short-span bridges, river crossings and railway and road way underpasses (Hurd et al. 1994). CSPI (2007) reported that CSP culverts were considered as an excellent alternative to the bridges replacement with a cost saving of 51%. CSP culverts gain their capacity from the interaction between the steel plate cross-section and the surrounding backfill. These structures are constructed by bolting and assembling corrugated steel plates (CSP) segments together to form different configurations (e.g., a pipe shape, an arch shape, or a box shape) (Newhook et al., 2010). Figure 1 shows the different configurations of CSP culverts.

In middle of the 19th century, Meyerhof & Fisher (1963) and White & Layer (1960) started utilizing the ring compression theory for design purposes. Duncan (1976); and Katona et al. (1976) evolved the finite element (FE) analysis as a powerful analytical tool to investigate the culverts' behavior deeply. Since the early of the 20th century, many full field tests were conducted (e.g., Elshimi, et al. 2013; Vallée 2015; and McGrath et al. 2002) to develop and enhance many design codes including the CHBDC. As declared above, many studies have been carried out to understand the static behaviour of buried CSP culverts. While, there were few researches on the seismic response of corrugated steel culverts.

Che et al. (2006) used a shaking table tests to study the seismic behavior of CSP closed culverts. a scaled down 1g tests were conducted using a tank with a 1200 mm width, 800 mm length and a 1000 mm depth. Elliptical culvert was instrumented to measure the change of internal forces during the earthquake loading condition. The simulated earthquake signal in the test represented Hyogoken-nanbu earthquake in Kobe, Japan that had a magnitude of 6.9 and peak ground acceleration (PGA) of 0.8g. The study explained that due to the subjected earthquake loading conditions, the corrugated culvert model exhibited large bending strains, but they did not exceed the allowable strain of the structure. Moreover, the normal strains were negligibly small. In this study, it can be seen that the tested culvert was a closed bottom culvert. Therefore, its seismic performance cannot be used to explain the seismic behavior of CSP arch open bottom culverts.

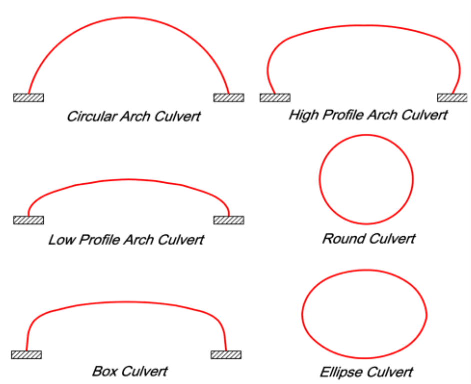

Figure 1: Different shapes of steel profiles (Reproduced after Beben, 2009)

Moreover, 1g shaking table tests on reduced scale models cannot model the stress-strain behavior due to the low stress field in the soil container. Thus, these tests are mostly used for clarifying the complex behavior of geotechnical structures subjected to seismic motions only.

Many other studies have also been conducted on the seismic design of underground culverts and pipelines (e.g. Hindy and Novak, 1979; NCHRP 611, 2008; and El Naggar, 2014). However, all of the above studies examined closed, relatively small pipes.

CHBDC depends on the study by Byrne et al. (1996) to calculate the CSP culvert's internal forces due to earthquakes (i.e., the bending moment and the thrust forces) by multiplying the static internal forces by $(1 + A_V)$. where, A_V is the vertical acceleration ratio and can be taken as two-thirds of the PGA of the specific region. However, this study did not account for the different site conditions and it used concrete arch culverts which have different stiffness range than that of CSP culverts and their material behavior is also different than that of the steel. Hence this study is deemed not so suitable to represent the behavior of CSP culverts specially under earthquake loading conditions.

A Numerical study by New Zealand Transport Agency (2008) was performed to examine the seismic behavior of CSP culverts. This study explained that the culvert internal forces experienced a notable increase due to the earthquake loading conditions. It was confirmed by Mahgoub and El Naggar (2017) that in low profile culverts located in the city of Victoria, the culvert's springline exhibited a significant increase in the internal forces and hence the simplified seismic design procedure of the CHBDC was found to be inappropriate for cities of high seismicity such as Victoria.

The main focus of this paper is to investigate the seismic performance of large span steel corrugated plate culverts in Eastern Canada. In general, there are seven main seismic regions in Eastern Canada which are Eastern Northern Ontario, Southern Great Lakes, West Quebec, Charlevoix-Kamouraska, Lower St. Lawrence, Northern Appalachians and Laurentian Slope. Two cities were selected in this study to be representative of Eastern Canada, namely Ottawa in Ontario and Saint John in New Brunswick. Ottawa has its importance, as it is the capital of Canada and can be considered as a representative of medium hazard cities in Eastern Canada (PGA = 0.28 g at site type C). While Saint John could be considered the representative of the Maritimes zone with PGA = 0.14 g in site type C according to NBCC (2015).

In this study, finite element (FE) analyses were carried out to study the seismic performance of CSP large span culverts located in Ottawa and Saint John representing Eastern Canada on rock (site type A), and on dense sand (site type C) site conditions. Subsequently, the FE results were compared with the results obtained by the simplified equations of the CHBDC to assess its suitability.

2 METHODOLOGY

The seismic/ dynamic analyses on the considered CSP culvert started with performing site response analyses to select the proper scaled artificial time histories that represent the seismic hazard in Ottawa and Saint John. Then, full dynamic finite element (FE) analyses using the scaled earthquake time histories in different site conditions (A and C) were developed to investigate the seismic performance of the CSP culverts in Eastern Canada.

3 SITE RESPONS ANALYSIS

Atkinson and Boore (2006); and Atkinson (2009) simulated a syntactic time history records to be compatible with the targeted UHS (Uniform Hazard Spectrum) for the eastern cities according to NBCC (National Building Code of Canada). Atkinson (2009) reported that the simulated records are developed for Eastern Canada according to the site condition, earthquake magnitude and the fault distances. Atkinson developed 5 sets for different earthquake magnitudes (i.e., M 6 and M 7). Every set has 45 random records at different fault location for every site condition to represent the earthquake in specific region/ site. Earthquakes (M 6) that were simulated in this study should match the short period end of the desired UHS, as per Atkinson (2009). These records were used in this study to select the proper time history records for the city of Ottawa and Saint John in Canada for sites A (rock) and C (dense sand).

3.1 ID Finite element analysis

Figure 2 shows the geometry of the 1D FE model. A 40 m of homogeneous soil was utilized to simulate the soil medium. 45 records were tried at the bottom of the model to select the most appropriate signal among them. The spectrum was estimated at the ground surface and was compared and scaled to the targeted spectrum in NBCC 2015.

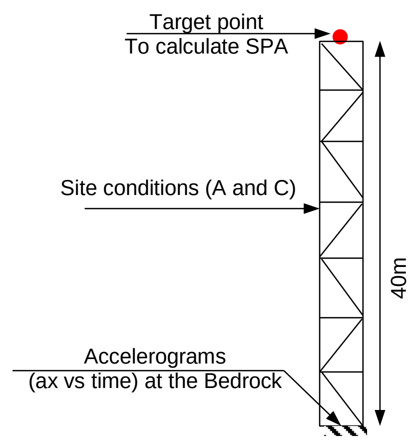

Figure 2: The developed geometry of the 1D FE analysis

3.2 FE mesh

15-noded triangular plain strain elements were utilized from the PLAXIS library to discretize the soil continuum. (Kuhlemeyer & Lysmer, 1973) proposed equation (1) to adjust the FE mesh dimension accordingly in order to prevent any seismic waves reflection during the analysis.

$\lambda/8 = (Vs,min)/(8 * f\max)$ [1]

where V_{smin} is the lowest shear wave velocity in the soil profile and f_{max} is the maximum frequency component of the inserted wave which can be calculated from Fourier spectrum analysis.

3.3 Soil model

Table 1 shows the soil parameters that were used in the analysis. The soil was modeled using the Hardening Soil Model with Small Strain (HS-Small) from the PLAXIS library. This model was developed by Benz (2006) to simulate the behavior of stress-dependent soils and accounts for the linear elastic relationship between the stress and strain in the first range of the small strain. The used parameters for the study were chosen according to reasonable ranges given in the literature (e.g., Bowels, 1997; Das and Sivakugan, 2016) and from the authors experience. The same parameters for the dense sand (site C) was used for the backfilling parameters.

Table 1. The soil parameters used

Material	φ'	E	E_{oed}	E_{ur}	ɣ	c
	°	MPa	MPa	MPa	kN/m^3	kPa
*Rock (A)	45	300	240	900	22	400
Dense sand (C)	42	50	32	120	20	0

*Assumed to represent the strong rock behavior with minor and negligible deformations

3.4 Damping

In PLAXIS, two targeted frequencies should be calculated to identify the damping matrix for any material; F_1 and F_2. F_1 is the fundamental frequency of the material. F_2 is the closest integral odd number given by the ratio between the fundamental frequency of the input signal (By Fourier Spectrum) and the fundamental frequency of the soil layer (Hudson, Idriss & Beirkae, 1994).

3.5 Boundary conditions

Tide degree of freedom was selected from the PLAXIS options to simulate the one-dimensional wave propagation, in order to connect the model's sides together and have the same displacement.

3.1.1 Results of the site response analysis

Figures 3 and 4 shows the scaled simulated response spectrum (SPA) of the scaled records for the cities of Ottawa and Saint John for sites A and C, respectively. It can be seen from the figures that the simulated response spectrums were compared with the targeted spectrum in NBCC 2015 within the period ranges of interest (0.1 to 1.0 sec.) as recommended by Atkinson (2009). The records' numbering is according to the available data in (Atkinson 2009).

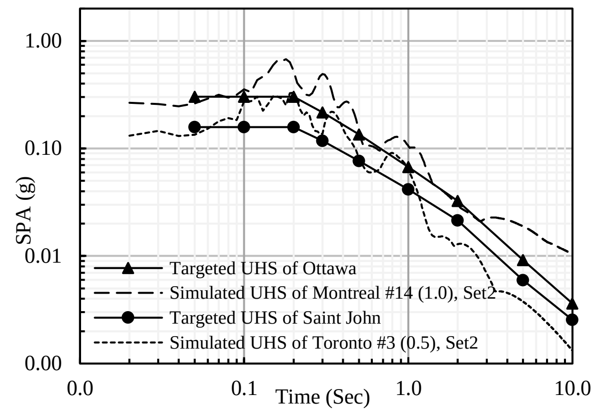

Figure 3. Targeted NBCC 2015 spectra compared to selected scaled simulated records in site A conditions

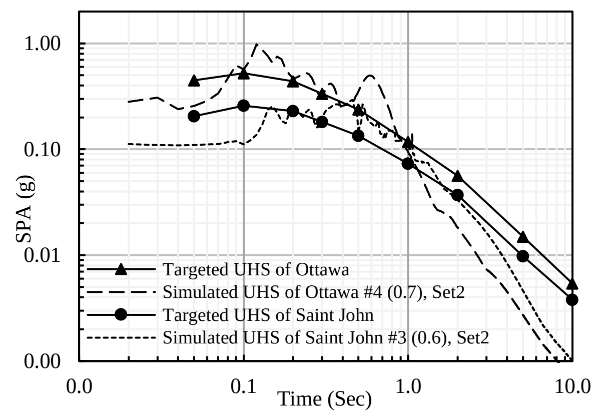

Figure 4. Targeted NBCC 2015 spectra compared to selected scaled simulated records in site C conditions

4 DEVELOPMENT OF FE MODEL FOR CSP CULVERT

Four FE models were developed using PLAXIS 2D. Two models represented the CSP culverts in Ottawa in sites A and C, while the other two models were developed for Saint John in the same site conditions. Figure 5 illustrate the culvert's geometry that was used in the analysis. It can be seen that the culvert is semi-circular with an 8.0 m diameter and a 4.0 m height. All of the simulated models have 0.9 m cover above the crown. Shallow corrugated cross-section was used in the analysis of 51 mm amplitude with a period of 152 mm. Table 2 shows the parameter used for the corrugated steel plate in the analysis.

HSM-Small was used to simulate the soil properties. Moreover, the assumptions for the formulation of the FE mesh in Section 3.2 were followed in the full model. The model comprised approximately 10,000 elements. In addition, free field boundary conditions were assigned to the model's lateral boundaries in the dynamic calculations with fully fixation at the bottom. The model lateral dimension was 150 m (about 19 times of the culvert diameter) to prevent any reversal propagations of the seismic waves. While, the total depth of the FE model was

40.0 m. Figure 6 displays the developed FE model with the different component.

The reinforced concrete footings were modeled as a volume element using an elastic material model and the used parameters are shown in Table 2. Additionally, the steel plates were modelled using isotropic elastic plate elements. Interface elements were placed between the CSP arches and the backfilling layers and between the subsurface soil and the concrete footings. The roughness of the interaction was modeled by utilizing a strength-reduction factor at the interface R_{inter} = 0.67. The base plate, which is used to fix the steel arches to the concrete footing, was a horizontal steel plate with 5 mm thickness and 300 mm width and anchored to the footing.

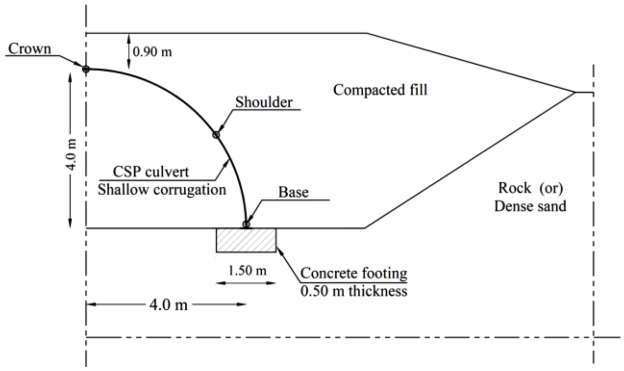

Figure 5. Steel arch geometry for this study

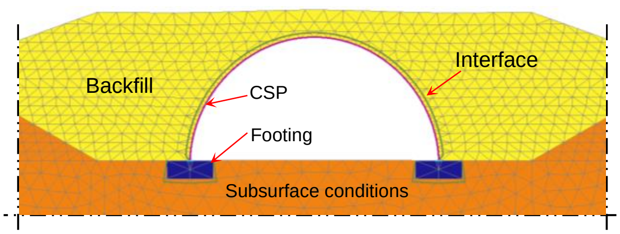

Figure 6. Mesh formulation

Table 2. Steel and concrete properties

Material	E (GPa)	Passion's Ratio	Inertia (mm^4/mm)
Concrete (Foundation)	30	0.15	----
Steel (Shallow corrugation)	200	0.3	2079.8

4.1 Construction sequence

The staged construction technique was used to simulate the construction process. The cut and cover method is typically used for these buried arches. In this construction method, the first stage involved excavating a trench to the foundation level. Then, the base foundations were placed followed by erection of the CSP culvert. The granular backfill was then placed in 0.3 m lifts till 0.90 m above the crown (4.9 m total height). Finally, the selected scaled signals were applied to the model's bottom boundary.

5 RESULTS AND DISCUSSION

In this section, the results of the finite element models are presented including the culvert deformations and internal forces and compared with the results of the simplified equations in the CHBDC under static and seismic loading conditions. Three main points (Crown, shoulder and base) were selected to present the obtained bending moments, and normal forces (Thrust).

Figure 7 shows the internal forces obtained from the FE models (i.e., the thrust forces and bending moments) for sites A and C under static conditions, compared to the results by the simplified equations of the CHBDC. Only the maximum thrust forces at the base and the bending moment at the crown were given by the CHBDC equations. As there are no considerations were proposed by the CHBDC for calculating the different internal forces in various locations around the culvert. It can be seen from the figure that the calculated values by the CHBDC's equations were identical in the different site conditions. While, there was a slight difference between the deducted results from the FE models for sites A and C, respectively. That is because the footings in the models in site C conditions exhibits 5 mm settlement, while the footings for the models in site A conditions did not have any movement.

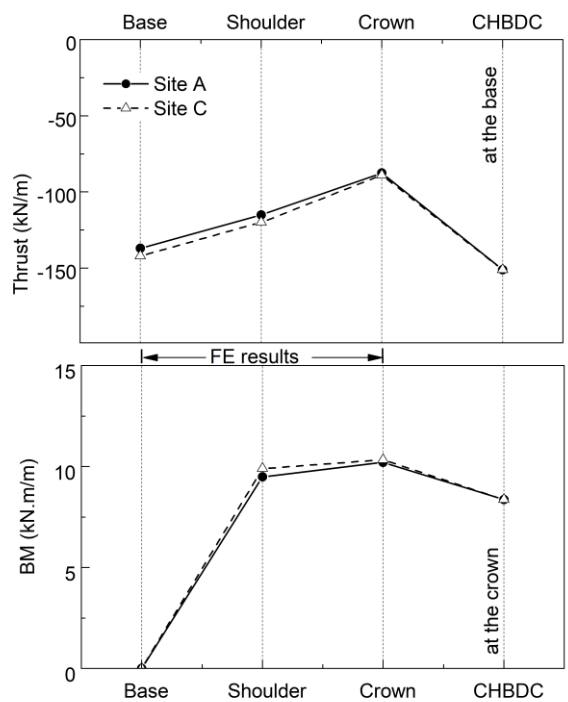

Figure 7. Internal forces under static conditions

Figures 8 and 9 illustrate the effect of the earthquake on the CSP culvert's internal forces (thrust forces and bending moments) for the city of Ottawa for the site type A throughout the duration of the earthquake. Figure 8 demonstrates that within the earthquake the thrust forces at the base increased from about 137 kN/m to 173 kN/m at the 21.86 seconds of the earthquake signal. However, the moment forces stabilized at almost zero as shown in Figure

9. In addition, Figure 9 shows that the maximum bending moment was found at the culvert's crown and was 12.18 kN.m/m at 23.76 seconds. The same procedures were followed to deduct the change of culvert's internal forces during the earthquakes in the different models. Subsequently, the maximum bending moments with accompanying thrust forces were plotted at different locations around the culvert (i.e., base, shoulder, and crown) in Figures 10 and 11.

Figure 10 shows that there was no significant difference between the FE results for the earthquakes in Ottawa in site types A and C, as the footing settlement for them was trivial (0, 7.5 mm). Moreover, it can be seen from this figure that the maximum thrust forces by the CHBDC's equations were -177 kN/m and -180 kN/m for sites A and C, respectively. While, the maximum thrust forces from the FE results were -173 kN/m and -178 kM/m at the culvert base. In addition, the maximum bending moments by the FE modeling that were encountered at the culvert crown were 12.1 kN.m/m and 11.6 kN.m/m for site types A and C, respectively. However, they were 9.77 kN.m/m and 9.92 kN.m/m by the CHBDC's equations. This increase in the bending moments did not lead to a significant increase in the culvert internal stresses as shown in Figure 12.

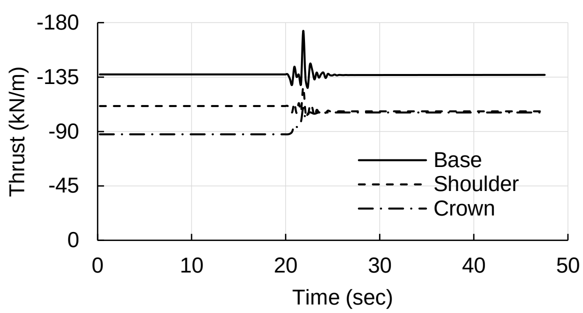

Figure 8. Thrust forces throughout the duration of an earthquake, Ottawa-site A

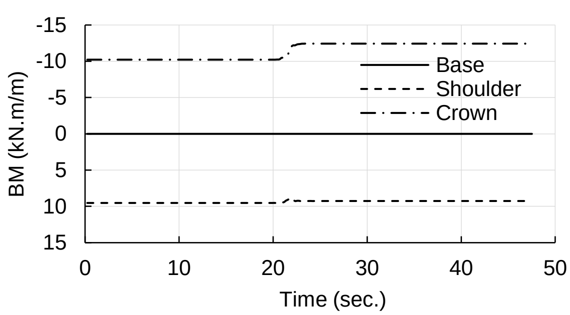

Figure 9. Bending moment throughout the duration of an earthquake, Ottawa-site A

Likewise, Figure 11 shows the comparison between the FE results and the results obtained by the CHBDC's equations for the city of Saint Johns for site types A & C. The same trend was found in these results, as the maximum thrust forces were found at the culvert's base. These forces by the CHBDC's equations were more than the deducted ones from the FE models by 5% and 2% for site types A and C respectively. However, the bending moments at the culvert's crown from the FE models were more than the calculated moments by CHBDC's equations by 17% and 15% for site types A and C. While, as concluded before this slight change did not exhibit a notable increase in internal stresses as shown in Figure 12. The internal forces in the models that were developed in Ottawa are more than the models in Saint Johns, as the seismic hazards in Ottawa is higher (PGA in Ottawa = 0.28 and in Saint Johns = 0.14).

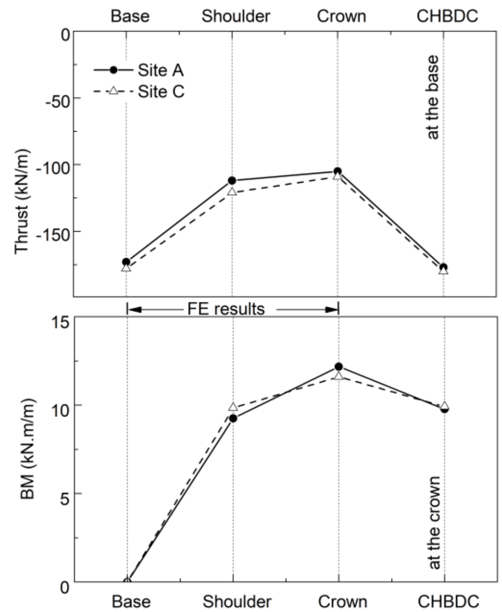

Figure 10. Maximum internal forces under seismic conditions for Ottawa

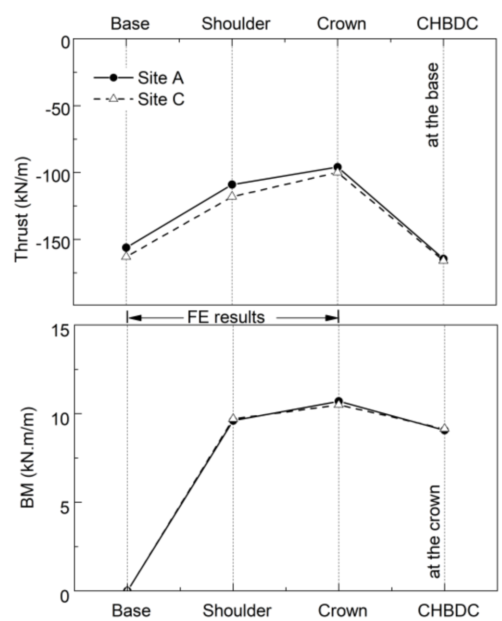

Figure 11. Maximum internal forces under seismic conditions for Saint John

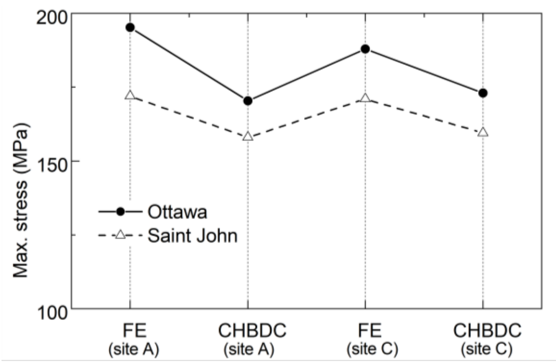

Figure 12. Maximum culvert's stresses in different cities and site conditions.

6 SUMMARY AND CONCLUSION

Detailed two-dimensional finite element models were developed to investigate the performance of CSP culverts under both static and seismic loading conditions in Eastern Canada. The study involved developing models that simulated the construction sequence and accounted for the soil-structure interaction including the effects of the compaction of the backfill around the culvert. The following conclusions can be drawn based on the results of the study:

- There is an acceptable match in the deducted thrust forces and bending moments from the FE models to the results of the CHBDC's equations in strong soil types (i.e., rock, dense sand) in Eastern Canada.
- Under the seismic conditions, The CHBDC's equations overestimated the maximum thrust forces. While, the obtained bending moments from the FE models are larger than the ones calculated by the code.
- This increase in the calculated bending moments did not lead to a significant increase in the culvert internal stresses.
- Hence, the simplified CHBDC equations can be used safely for cities located in zones of low seismicity.

ACKNOWLEDGEMENTS

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