12TH CANADIAN CONFERENCE ON EARTHQUAKE ENGINEERING
JUNE 17-20, 2019
CHÂTEAU FRONTENAC
QUÉBEC, QC.
12ÈME CONFÉRENCE CANADIENNE DU GÉNIE PARASISMIQUE
17-20 JUIN 2019
CCEE 2019

# Liquefaction design criteria under consideration by British Columbia Ministry of Transportation and Infrastructure

Alireza Ahmadnia[1], Kevin Baskin[2], Sarah Gaib[3]

[1] Ph.D., P.Eng. Senior Seismic Bridge Engineer, BC Ministry of Transportation and Infrastructure-Vancouver, BC, Canada.
[2] M.Sc, P.Eng. Bridge Engineer, BC Ministry of Transportation and Infrastructure-Victoria, BC, Canada.
[3] M.Eng,P.Eng. Lead Geotechnical Engineer, Foundations, BC Ministry of Transportation and Infrastructure-Victoria, BC, Canada.

## ABSTRACT

The British Columbia Ministry of Transportation and Infrastructure (BC-MoTI) owns a large inventory of structures (such as bridges, tunnels, walls, high mast light poles). These structures, depending on their importance, are categorized as lifeline, major-route, or other. They are located in different BC seismic environments- seismically active areas in western BC as well as relatively inactive seismic areas in eastern BC. The Canadian Highway Bridge Design Code (CHBDC) and the BC Supplement to the CHBDC define criteria for the liquefaction potential of foundation soils and the impact of soil liquefaction on structures. The provided criteria for liquefaction susceptibility analysis and consequence analysis range from simplified analysis to rigorous analysis in which the level of effort, expertise, input parameters, and geotechnical investigations to do these types of analysis vary substantially. However, the CHBDC wording for liquefaction analysis is general and does not indicate which method to use for which type of structures under which conditions. Since it is not the intent of codes to provide prescriptive direction on all input parameters, it is therefore left to the judgement of the designer and/or the Regulatory Authority to decide on the specific methodologies and inputs. This paper presents considerations for proposed revisions to the BC Supplement to CHBDC for liquefaction design of BC-MoTI structures.

Keywords: Seismic, Liquefaction, Performance-based design, Liquefaction triggering and consequence analysis, Highway structures.

## INTRODUCTION

The seismic design of BC-MoTI structures (such as bridges, tunnels, walls, high mast light poles) is performed in accordance with the CSA-CHBDC S6-14 [1] ("CHBDC") and the BC Supplement to the CHBDC S6-14 [2] ("BC Supplement"). The CHBDC and BC Supplement define criteria for determining the liquefaction potential of foundation soils and the impact of soil liquefaction on structures. However, since it is not the intent of codes to provide prescriptive direction on all input parameters, there are areas where direction is not provided regarding methodologies and inputs. These areas include: requirements for liquefaction triggering analysis and liquefaction induced consequence analysis, ground motions and earthquake magnitude for liquefaction analysis, the impact of liquefaction on the structure, the design response spectrum for the structure on liquefiable soils, the combination of inertial and kinematic demands, and the design and analysis requirements of the structure on liquefiable soils. In order to provide clarity to the designer and standardize the process for liquefaction analysis and design of BC-MoTI structures, revisions are being considered for the BC Supplement.

In the following section, the current criteria in the CHBDC and BC Supplement regarding the liquefaction potential of foundation soils and the impact of soil liquefaction on structures is briefly overviewed and areas proposed for further direction regarding criteria are discussed. Thereafter, proposed revisions being considered for the BC Supplement wording are presented.

## CURRENT WORDING IN CHBDC AND BC SUPPLEMENT

### Liquefaction potential of foundation soil

According to the CHBDC Clause 4.6.6.1, the potential for liquefaction needs to be evaluated in the following three stages:

> (a) susceptibility analysis based on geologic age and depositional origin of soils, index properties and gradation data, and degree of saturation;

(b) analysis based on anticipated ground surface acceleration, cyclic shear stresses, and penetration resistance or shear wave velocity; and
(c) analysis based on site response modelling, site-specific ground motions, and appropriate analytical models of soil behavior.

Screening (stage (a) above) is the initial assessment of a site for liquefaction potential using information about groundwater table, age of soil (CHBDC Table C4.4), liquefaction hazard maps (GSC Open file 3511, 1998), soil type, soil density, and peak ground acceleration. The commentary on the CHBDC C4.6.6 provides a summary of preliminary screening of liquefaction susceptibility of soils. If the soil is judged to be susceptible to liquefaction, the analysis for the liquefaction triggering continues per stage (b) and (c). The next step requires the calculation of cyclic shear stresses for which there are many methods available. The analysis ranges from simple analysis such as "Simplified stress-based method analysis" using the Seed-Idriss simplified equation [3] to rigorous analysis such as effective stress analysis and less intensive analysis such as an equivalent-linear total stress analysis. The CHBDC does not provide direction for which method to use for which structures under which conditions.

**Impact of soil liquefaction on bridge foundation**

According to the CHBDC Clause 4.6.6.2, once the soil has been determined to be potentially liquefiable, one or more of the following mitigation measures are taken: (a) use of an appropriate foundation type; (b) implement soil improvement methods; and (c) design the bridge superstructure to withstand predicted foundation movements.

The design of the bridge to withstand liquefaction induced ground movement requires the prediction of forces and displacements generated by the lateral spreading or lateral flow. Several methods are available to estimate the liquefaction induced ground movements. Similar to the liquefaction triggering analysis, the methods range from simplified methods such as empirical or semi-empirical regression-based predictive relationships to rigorous analysis such as nonlinear finite element analysis and less intensive deformation analysis such as Newmark type analysis. The CHBDC does not provide direction for which methods to use for which structures under which conditions.

**Design requirements for structures on liquefiable soils**

In the CHBDC, liquefaction assessment is not dependent on whether performance-based design (PBD) or force-based design (FBD) is being used for structures. The CHBDC therefore allows the use of either FBD or PBD for sites on liquefiable sites. However, it is logical to use PBD for the structure since this method requires that it be demonstrated that the structure can tolerate the predicted ground movements and meet the required performance criteria. Using PBD, the structure needs to be designed so that the foundation movement due to the liquefaction-induced ground movement meets the BC Supplement performance criteria for foundations as which are shown in Table 1.

*Table 1. Foundation movement performance criteria from the BC Supplement*

| Minimal damage | Repairable damage | Extensive damage | Probable replacement |
|---|---|---|---|
| Foundation movements shall be limited to only slight misalignment of the spans or settlement of some piers or approaches that does not interfere with normal traffic, provided that no repairs are required. | Ground deformation shall be mitigated such that permanent foundation offsets are small and repair objectives can be met. Foundation offsets shall be limited such that repairs can bring the structure back to the original operational capacity. | Foundation lateral and vertical movements must be limited such that the bridge can be used by restricted emergency traffic. Foundation offsets shall be limited such that repairs can bring the structure back to the original operational capacity. | Foundation movement shall not lead to collapse nor prevent evacuation. |

With performance-based design, in-ground plastic hinges in piles and movement of slopes, embankments and retaining walls are permitted provided the desired performance criteria of the structure is met. The commentary to the BC Supplement refers designers to the MCEER/ATC-49 [4] document as a guideline in determining performance limits for piles. ATC-49 provides the maximum rotational capacity for steel and concrete piles for significant damage and minimal damage performance levels as shown in Table 2. Analysis is then needed to relate rotational capacities to concrete and steel strain limits. ATC-49 does not provide the rotational capacity for repairable damage nor probable replacement.

**Design spectral values for structures on liquefied soils**

The CHBDC identifies liquefiable soils as site class F and requires site-specific evaluation. In addition to the requirement of the CHBDC, the current BC Supplement has additional wording which requires inertial demands be established based on the Geological Survey of Canada (GSC) response spectra for site condition as per shear wave classification or by using spectra

*Table 2. MCEER/ATC-49 Pile performance limits (plastic rotation in radians)*

| | Minimal damage | Significant damage |
|---|---|---|
| In-ground plastic hinge | | |
| Steel | .005 | .01 |
| Concrete | .005 | .02 |

from site response analysis. The site response analysis for Site Class F may be 1D equivalent linear analysis, 1D nonlinear analysis, or 2D nonlinear analysis. Currently, neither the BC Supplement nor the CHBDC specify which methods to use to obtain design spectral values for liquefiable soils. Also, these documents do not specify whether the spectral values are to be based on liquefied or non-liquefied soil properties.

### Combining seismic inertial loading with kinematic loading

The current CHBDC does not provide any guidance on how to combine the kinematic and inertial demands. The BC Supplement requires the effects of inertial loading to be combined with liquefaction induced kinematic loading using 50% inertial demands plus 100% kinematic demands. Also, the BC Supplement requires a combination of inertial and kinematic demands where soil softening does not reduce the inertial effect.

## PROPOSED LIQUEFACTION DESIGN REQUIREMENTS UNDER CONSIDERATION FOR THE BC-SUPPLEMENT

The BC-MoTI structures have different importance classifications and are designated either as lifeline, major-route, or other structures. In addition, BC seismicity varies tremendously from highly active areas in western BC to relatively inactive areas in eastern BC. Even within western BC, the seismic hazard can vary substantially. The seismic environments of the southwest part of BC is affected by Crustal, In-slab, and Interface earthquakes. Each of these events can have different liquefaction results. Interface earthquakes may produce large magnitude earthquake subjecting the soil to a large number of cycles while Crustal or In-slab earthquakes may produce larger ground shaking with a smaller number of cycles. In addition, the overall level of effort, required level of geotechnical investigation, experience in modelling and interpreting the results, and required number of inputs such as ground motions for liquefaction triggering, deformation analyses, and ground response analysis vary substantially between different liquefaction analysis methods. To provide more direction to designers and standardize the liquefaction analysis methodologies, the BC-MoTI is considering minimum liquefaction analysis and design requirements for its structures based on the importance of the structure and the seismic hazard at the site. In the next section, proposed criteria being considered for revisions to the BC Supplement is presented. These criteria have been implemented on a trial basis in several projects and are currently under review for addition to the BC Supplement.

### Liquefaction triggering analysis

Liquefaction triggering of susceptible soils involves comparison of the anticipated shear stress produced by ground shaking with the liquefaction resistance of the soil. Table 3 shows proposed minimum liquefaction triggering analysis requirements. Table 3 is organized based on the seismic performance category (SPC) and the structure importance. SPC is defined according to the CHBDC Clause 4.4.4.

*Table 3. Proposed BC MoTI Liquefaction triggering analysis*

| SPC | Lifeline bridges | Major route bridges | Other bridges |
|---|---|---|---|
| 1 | Routine | Simplified | Simplified |
| 2 | Routine | Simplified | Simplified |
| 3 | Routine or Rigorous * | Routine or Rigorous * | Simplified or Routine ** |

*BC-MoTI consent and independent peer review are required. **BC-MoTI consent is required.

Simplified analysis is the most common method for the liquefaction triggering analysis. This type of analysis involves comparing the earthquake induced cyclic shear stress (CSR) to the cyclic resistance ratio (CRR). The Commentary on the CHBDC C.4.6.6.1 provides procedures such as Youd et al. (2001) [5] or Idriss and Boulanger (2008) [6] for the assessment of liquefaction triggering. The assessment for soil liquefaction needs to be based on using a single consistent methodology throughout the analysis. The Commentary on the CHBDC states that a factor of safety varying between 1.0 and 1.3 is commonly used in the liquefaction assessment. BC-MoTI is proposing to use a factor of safety of 1.0. For simplified analysis, the following criteria regarding acceleration, earthquake magnitude and geotechnical understanding is proposed:

- Peak ground acceleration is determined using the site amplification factors F(PGA) given in Table 4.8 of the CHBDC adjusted for site class using non-liquefied soil properties. In SPC=1, PGA corresponds to the 2475-year return period event.
- Magnitude is the mean earthquake magnitude obtained from the de-aggregation of PGA from Natural Resources Canada (NRCan).
- At least a typical degree of site understanding of ground properties and geotechnical properties as defined in the CHBDC Clause 6.5.3 is required.

Routine analysis would require the use of a 1D equivalent-linear or nonlinear ground response analysis programs (such as SHAKE [7]) to obtain CSR. Input for the ground motion analysis is described in the "Ground motion and magnitude for liquefaction analysis" section of this paper. For routine analysis the following is proposed:

- Simplified analysis is also conducted as well for comparison purposes.
- A high degree of site understanding of ground properties and geotechnical properties as defined in the CHBDC Clause 6.5.3 is required.

Rigorous analysis would include the use of a 2D or 3D nonlinear effective stress analysis or nonlinear total-stress analysis programs (such as FLAC [8]). In these programs, pre-triggering, triggering, and post-triggering aspects of soil liquefaction response are included in one analysis. Also, the effects of soil-structure interaction and ground improvement can be included. Input for the ground motion analysis is described in the "Ground motion and magnitude for liquefaction analysis" section of this paper. For rigorous analysis the following is proposed:

- This type of analysis needs to be consented to by the BC-MoTI on a project by project basis.
- An independent peer review is required. The independent peer review should be undertaken by Professional Engineers recognized as leading experts, and suitably qualified by education and/or experience.
- This type of analysis must address pre-triggering, triggering, and post-triggering aspects of liquefaction.
- Routine 1D analysis is also conducted for comparison purposes.
- A high degree of site understanding of ground properties and geotechnical properties as defined in the CHBDC Clause 6.5.3 is required.

**Liquefaction-induced consequence analysis**

If liquefiable soils are confirmed, then analysis for estimating the ground movement due to liquefaction is required. Table 4 presents the proposed minimum analysis requirement. Table 4 is organized based on SPC and the structure importance.

*Table 4. Proposed BC MoTI Liquefaction-induced consequence analysis*

| SPC | Lifeline bridges | Major route bridges | Other bridges |
|---|---|---|---|
| 1 | Simplified | Simplified | Simplified |
| 2 | Simplified or Rigorous * | Simplified | Simplified |
| 3 | Simplified or Rigorous * | Simplified or Rigorous * | Simplified |

*BC-MoTI consent and independent peer review are required.

Simplified analysis would include empirical-based approaches such as Youd (2002) [9], semi-empirical approaches such as Faris (2007) [10], and Newmark-based analysis such as Kavezanjian (2011) [11]. The displacement obtained from the simplified analysis can be highly variable and depends upon local topography, soil stratigraphy, material properties, and ground motion. Engineering judgement should be used to determine lateral displacement values to be used in the assessment of the bridge performance. The assumptions, limitations, and applicability of the selected methodologies should be assessed. It is recommended that for estimates of lateral spread displacement using the simplified method, at least two approaches should be selected to evaluate a likely range of potential lateral displacements.

Rigorous analysis would require 2D or 3D nonlinear effective stress analysis or nonlinear total-stress analysis using the following proposed criteria:

- This type of analysis needs to be consented to by the BC-MoTI on a project by project basis.
- An independent peer review is required as described in "Liquefaction triggering analysis" section.
- This type of analysis must address pre-triggering, triggering, and post-triggering aspects of liquefaction.
- Simplified analysis is conducted for comparison purposes.
- A high degree of site understanding of ground properties and geotechnical properties as defined in the CHBDC Clause 6.5.3 is required.

### Ground motion and magnitude for liquefaction analysis

The ground motions used in routine and rigorous analysis need to be in accordance with the CHBDC Clause 4.4.3.6 and its commentary. These provide guidance on the selection, scaling and number of ground motions.

The BC-MoTI has developed site-specific ground motion records for several projects. The latest set of ground motion records were developed for the George Massey Tunnel (GMT) Replacement Project based on site-specific scenario horizontal acceleration response spectra considering scenario specific period ranges [12]. The ground motions developed for the GMT site were spectrally matched to target response spectra. In total 45 sets of earthquake motions were developed. This includes 5 sets of ground motions for each of the Crustal, In-slab, and Interface scenario earthquakes for the 475, 975 and 2475 return periods. Each set consists of three ground motion records spectrally matched to target response spectra resulting in a total of 135 individual acceleration time-histories. Median spectra shape, as an alternative approach for defining site-specific response spectra, was used for target response spectra.

Where the BC-MoTI considers these GMT ground motions are applicable at different project sites, the BC-MoTI has provided these records to the designers. These then need to be uniformly scaled to the site-specific horizontal peak ground acceleration for the structure's site. When using the GMT records, the earthquake magnitude to be used for Routine liquefaction assessment would be the mean magnitude at the site based on PGA de-aggregation of seismic hazard for Crustal and In-slab earthquakes and M = 8.4 for Interface earthquakes. Magnitude M = 8.4 corresponds to the mean of the magnitude of the five ground motion records selected for the site-specific GMT interface event.

In cases where the GMT ground motions are not considered to be applicable to different project sites, the target response spectrum for ground motion selection needs to be the uniform hazard spectrum as obtained from the GSC. Use of site-specific scenario or alternative response spectra as a target response spectrum is only to be used when consented to by the BC-MoTI on a project by project basis. Finn et al [13] demonstrated that the mean magnitude and magnitude de-aggregation methods gave the same results for liquefaction potential and lateral displacements. It is proposed that this study can be extended to use for Routine analysis when ground motions are selected to match uniform response spectra selected as target response spectra. Therefore, the magnitude for liquefaction assessment would be the mean earthquake magnitude of the combined hazards as obtained from the de-aggregation of PGA from NRCan. Alternatively, magnitude de-aggregation of PGA using mean value for each scenario earthquake can be used for Routine liquefication assessment with scaling the analysis result by the contribution factor of each scenario earthquake to the PGA value.

### Impact of liquefaction on the structure

When considering the effects of liquefaction, the structure is designed for both liquefied and non-liquefied conditions. Firstly, the structure is designed for the non-liquefied condition (item (a) below). The non-liquefied case will control the loads applied to the structure. Then the structure is checked for the liquefied condition (item (b) below) which will control the deformations in the structure. For both item (a) and item (b), the input design response spectrum or earthquake motion is based on ground response analysis using the non-liquefied soil condition. AASHTO LRFD [14] uses this same approach. Youd and Carter (2005) [15] suggest that at periods greater than 1 second, it is possible for liquefaction to result in higher spectral accelerations than the equivalent non-liquefied configuration. Therefore, item (c) is added for structures with a period greater than 1 second to check the possibility of whether the liquefaction can result in higher spectral accelerations than the equivalent non-liquefied condition. The following provides the proposed wording for the analysis and design of the structure where liquefaction will occur around it:

(a) Non-liquefied condition: The structure shall be designed for inertial loading, assuming no liquefaction or cyclic mobility occurs, using the design response spectrum for Elastic Dynamic Analysis (EDA) and Inelastic Static Pushover Analysis (ISPA) or earthquake ground motions for Nonlinear Time History Analysis (NTHA) analysis appropriate for the site soil conditions in a non-liquefied state.

(b) Liquefied condition: The structure as designed per condition described in Item (a) shall be checked using resistance parameters such as P-y curves, modulus of subgrade reaction, and/or t-z curves appropriate for liquefiable or cyclically mobile soil conditions. The design response spectrum for EDA and ISPA analysis or earthquake ground motions for NTHA shall be the same as that used in a non-liquefied condition.

(c) The designer shall verify whether soil liquefaction will result in higher spectral acceleration than the equivalent non-liquefied case by using the liquefied soil response spectra. The design shall be based on the worst case.

### Design requirements for structures on liquefiable soil

The proposed wording for the analysis requirement of bridges on liquefiable soil is as follows:

All bridges on Site Class F liquefiable soil in Seismic Performance Categories 2 and 3 shall use performance-based design. If lateral flow or lateral spreading is predicted to occur, the following requirements for the foundations shall be met:

- Design the foundations to resist the forces generated by liquefaction induced ground movements.
- If the foundations cannot be designed to resist the forces, assess whether the Structure is able to tolerate the anticipated movement and meet the performance criteria of the structure. Controlled plasticity (in-ground plastic hinging) may be relied upon in piles for load cases with liquefaction induced kinematic loading provided that requirements in the table below are met.
- If the foundations cannot meet the requirements in Table 5, then mitigation measures are required to minimize the movement to tolerable amounts such that the performance criteria are satisfied.

*Table 5. Proposed BC-MoTI Maximum allowable Steel and Concrete Strain for Pile In-Ground Plastic Hinges.*

| Type | 10% in 50 years | | | 5% in 50 years | | | 2% in 50 years | | |
|---|---|---|---|---|---|---|---|---|---|
| | **Damage** | **Concrete Strain** | **Steel Strain** | **Damage** | **Concrete Strain** | **Steel Strain** | **Damage** | **Concrete Strain** | **Steel Strain** |
| **Lifeline** steel pile | Minimal | .004 | $\varepsilon_y$ | Minimal | .004 | $\varepsilon_y$ | Repairable* | 0.4 $\varepsilon_{cu}$ | .01 |
| **Lifeline** concrete pile | Minimal | .004 | $\varepsilon_y$ | Minimal | .004 | $\varepsilon_y$ | Repairable* | .006 | .01 |
| **Major-route** steel pile | Minimal | .004 | $\varepsilon_y$ | Repairable* | 0.4 $\varepsilon_{cu}$ | .01 | Extensive** | 0.75 $\varepsilon_{cu}$ | .02 |
| **Major-route** concrete pile | Minimal | .004 | $\varepsilon_y$ | Repairable* | 0.006 | .01 | Extensive** | 0.75 $\varepsilon_{cu}$ | .02 |
| **Other** steel pile | Repairable | 0.4 $\varepsilon_{cu}$ | .01 | Extensive** | 0.75 $\varepsilon_{cu}$ | .02 | Probable replacement | $\varepsilon_{cu}$ | .05 |
| **Other** concrete pile | Repairable | 0.006 | .01 | Extensive** | 0.75 $\varepsilon_{cu}$ | .02 | Probable replacement | $\varepsilon_{cu}$ | .05 |

$\varepsilon_y$: yield strain and $\varepsilon_{cu}$: ultimate confined strain

*For repairable damage, one of the following conditions shall be met

- The Designer shall submit a repair scheme demonstrating that normal service can be restored within a month. The repair scheme shall be subject to acceptance by the BC-MoTI. Installation of devices that permit post-EQ assessment of in-ground plastic hinge performance is required.
- In cases where the Designer has determined that repairs are not required to restore normal service, the Designer shall demonstrate to the satisfaction of the BC-MoTI that the damaged bridge has sufficient capacity for all design loads and load combinations specified in the CHBDC for new bridges.
- If the above conditions cannot be met, Minimal damage strain limits shall be used as per the BC Supplement.

** For extensive damage, the Designer shall submit a repair scheme demonstrating to the satisfaction of the BC-MoTI that damage does not preclude return of full service. Repairs to restore the bridge to full service might require bridge closure.

Where in-ground pile plastic hinges are used, explicit pile foundation modelling is required with ISPA and NTHA. The explicit foundation modelling shall include a representation of each individual pile with distributed soil supports over the entire pile length.

The philosophy in defining criteria for in-ground plastic hinges is to avoid the difficulty of post-earthquake inspection and high costs associated with repair of damaged foundations. The proposed strain limits in Table 5 have been determined by reviewing the BC-MoTI's past projects (such as seismic design criteria of the main span of Port Mann Bridge) and other jurisdictions' criteria such as the Port of Long Beach Wharf Design Criteria [16]. Since in-ground plastic hinges are not readily inspectable post-earthquake, installation of devices that permit post-earthquake assessment, for example installation of inclinometer tubes, is proposed. These types of devices would help in the post-earthquake evaluation of in-ground plastic hinging.

**Design response spectra for structures on liquefied soil**

Where site-response analysis of the structure is required for liquefaction assessment, the design response spectrum for EDA and ISPA and earthquake ground motions for NTHA need to be obtained from the result of the site response analysis using non-liquefied soil properties. Site-specific response spectra from site response analysis shall not be less than 80% of the code-based spectra for the applicable site class using non-liquefied soil properties.

Liquefied soil response spectrum using the liquefied soil properties would only be used as described in item (c) under the section "Impact of liquefaction on the structure"

**Combining seismic inertial loading with kinematic loading**

The duration of ground motions and the resistance of the soil to liquefaction are two important factors which affect the timing issue of inertial and kinematic demands and their combination. For instance, for long duration events such as Interface events located at sites with loose soils, liquefaction may be triggered at the beginning of shaking. In this case, the structure is likely to be subjected to high inertial demands while the soil is in a liquefied state. However, for short duration events such as Crustal events located at a site with dense soil, liquefaction may be triggered at the end of shaking. In this case, the structure is unlikely to be subjected to high inertial forces while the soil is in a liquefied state. Currently, there is no consensus within the engineering community of how to combine the inertial and kinematic effects. Table 6 presents some of the common approaches by different standards, guidelines, and researchers.

*Table 6. Inertial and kinematic demands combination*

| Document name | Inertial and kinematic loading combination |
|---|---|
| ATC-49 (2003) | No combination unless agreed to or directed otherwise by the owner. |
| AASHTO LRFD (2017) | No combination but recognizes for M>8 there is more likely strong shaking and liquefaction occur concurrently. |
| WSDoT (2013) [17] | 100% (kinematic)+ 25%(inertial) for sites where more than 20 percent of the hazard contributing to the peak ground acceleration is from an earthquake with a magnitude more than 7.5. |
| CALTRANS Guideline for liquefaction (2012) [18] | 100% (kinematic)+50%(inertial) for pile cap displacement<br>100%(kinematic) ±50%(inertial) for peak pile moment and shear |
| Ashford et al. (2011) [19] | For a typical design acceleration response spectrum having Sa(1s)/Sa(0s) = 0.5 – 1.6:<br>100% (kinematic)+64%(inertial) for pile cap<br>100% (kinematic)+36%(inertial) for superstructure |

The timing issue may be resolved by using rigorous coupled soil-structure analysis (direct method); i.e., finite element or finite difference modeling of the soil and structure together. However, this type of analysis is computationally intensive and requires considerable experience in geotechnical and structural finite element modeling. In lieu of this type rigorous coupled analysis, the decoupled soil-structure analysis is proposed by the BC-MoTI with a conservative approach of combining the inertial and kinematic demands. The proposed approach considers the seismic hazards at the structure site and the importance of the structure. The following is the proposed wording for the BC Supplement for combining inertial and kinematic effects. This wording is consistent with a draft of CSA S6-19 CHBDC that was issued for public comment in 2018.

- For structures in SPC=2 that are not classified as Lifeline, the effects of liquefaction or cyclic mobility-related permanent lateral ground displacements on bridge and geotechnical system performance shall be considered separately from the inertial evaluation of the bridge structure.
- All structures in SPC=3 and Lifeline bridges in SPC=2 shall consider the potential simultaneous occurrence of the inertial demands on the bridge and the kinematic demands on foundations from permanent lateral displacement of the soil. The effects of kinematic loading from inelastic ground deformations on the structure shall be evaluated and combined with the displacement and other effects of inertial loading using the combination of 100% kinematic demands ± 50% inertial demands. Inertial demands shall be obtained from worst case demands from items (a), (b), and (c) as described above in the "Impact of liquefaction on structure" section. In cases where soil softening does not reduce the inertial effect, then a special assessment shall be undertaken to develop an appropriate combination of inertial plus the applicable kinematic effects.

## CONCLUSIONS

In this paper, the current wording for the liquefaction design requirements per the BC Supplement and CSA-CHBDC-S6-14 are overviewed and proposed wording being considered for revisions to the BC Supplement are presented. The proposed wording gives more direction and guidance regarding liquefaction triggering analysis, liquefaction consequence analysis, the analysis and design requirement for structures on liquefiable soil, the combination of inertial and kinematic effects, and the design response spectrum for liquefiable soil.

## ACKNOWLEDGMENTS

The authors acknowledge and thank Upul Atukorala. Ph.D, P.Eng, from Golder, Uthaya Uthayakumar, Ph.D, P.Eng, from Stantec, Donald Gillespie, Ph.D, P.Eng. from Tetratech, Sharlie Huffman, P.Eng. from Huffman Engineering Ltd., Don Kennedy, P.Eng from Associated Engineering, and Bruce Hamersley, P.Eng. from Klohn Crippen Berger Ltd., and David Woolford, P.Eng. Senior Bridge Design and Construction Engineer, from BC-MoTI for their advice and assistance in the development of the proposed revisions to the BC Supplement.

## REFERENCES

[1] Canadian Standard Association - CSA (2014). *CSA-S6 : Canadian Highway Bridge Design Code*. Prepared by the CSA, Toronto, ON.

[2] *Bridge Standards and Procedures Manual-Volume 1-Supplement to CHBDC-S6-14.* (2016) Prepared by the Ministry of Transportation and Infrastructure.

[3] Task Force Report Geotechnical Design Guideline for Building on Liquefiable Sites in accordance with NBC 2005 for Greater Vancouver Region (2007)

[4] MCEER/ATC-49 (2003), *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges*.

[5] Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L., Harder, L.F., Jr., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson III, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B. and Stokoe II, K.H. (2001) . Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils” , *Journal of Geotechnical and Geo-Environmental Engineering, ASCE,* 127(10): 817‑833.

[6] Idriss, I. M. and Boulanger, R. W. (2008). Soil liquefaction During Earthquakes, EERI Monograph MNO-12.

[7] Schnabel, P.B., Lysmer, J. and Seed, H.B. (1972). SHAKE — A Computer Program for Earthquake Response Analysis of Horizontally Layered Site, Report No. EERC 72-12, Earthquake Engineering Research Center, University of California, Berkeley.

[8] FLAC- Explicit Continuum Modeling of Non-Linear Material Behaviour in 2D, Itasca Consulting Group INC. Minneapolis, USA.

[9] Youd, T.L., Hansen, C.M., and Bartlett, S.F. (2002). “Revised Multi-linear Regression Equations for Prediction of Lateral Spread Displacement”, *Journal of Geotechnical and Geoenvironmental Eng*.128(12), 1007-017.

[10] Faris, A.T., Seed, R.B., Kayen, R.E., and Wu, J., (2006). “A semi-empirical model for the estimation of maximum horizontal displacement due to liquefaction-induced lateral spreading”, 8th National Conference on Earthquake Engineering, EERI, San Francisco, CA.

[11] Kavazanjian, E., Wang, J.-N. J., Martin, G. R., Shamsabadi, A., Lam, I. (P.), Dickenson, S. E., and Hung, C. J., (2011), *Geotechnical Engineering Circular #3, LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations. Report No. FHWA-NHI-11-032*.

[12] *Technical Memorandum, Earthquake Scenario Spectra and Acceleration Time-Histories 2475-YR, 975-YR and 475-YR Return Periods for George Massey Tunnel Replacement Project, Delta, British Columbia* (2016).

[13] Finn W.D. L, Dowling J., Ventura, C.E (2016), “Evaluating Liquefaction Potential and Lateral Spreading in a Probabilistic Ground Motion Environment”, *Soil Dynamic and Earthquake Engineering*, 91, 202-208.

[14] AASHTO LRFD Bridge Design Specification, (2007)

[15] Youd, T.L. and Carter, B.L., (2005). “Influence of Soil Softening and Liquefaction on Spectral Acceleration”, *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, 131(7), 811-825.

[16] Port of Long Beach. (2007). Wharf Design Criteria, Version 1.0. Port of Long Beach, Long Beach, CA.

[17] WSDOT Geotechnical Design Manual (2013)- Prepared by Washington Department of Transportation.

[18] Caltrans, (2012), Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading, Sacramento, CA.

[19] Ashford, S. A., Boulanger, R. W., and Brandenberg, S. J., (2011), Recommended Design Practice for Pile Foundations in Laterally Spreading Ground, PEER Report 2011/04, Berkeley, CA.