The 11th Canadian Conference on Earthquake Engineering
Canadian Association for Earthquake Engineering

SEISMIC DESIGN OF BRIDGES IN BRITISH COLUMBIA: TEN-YEAR REVIEW

Jamie McINTYRE
Structural Engineer, Hatch Mott MacDonald, Vancouver Canada
jamie.mcintyre@hatchmott.com

Marc GÉRIN
Consultant, Ottawa Canada
marcgerin@gmail.com

Casey LEGGETT
Structural Engineer, Hatch Mott MacDonald, Vancouver Canada
casey.leggett@hatchmott.com

ABSTRACT: Seismic design of bridges in British Columbia has evolved significantly in the last ten years. Developments have comprised three major changes in seismic design practice: (1) improved understanding of seismic hazard—including raising the design earthquake from a 475-year return period to 2475-year return period and better knowledge of the contribution of the nearby Cascadia subduction zone; (2) a shift to a performance-based design philosophy with emphasis on improved post-earthquake performance—including multiple service and damage objectives for multiple levels of ground motions; and (3) increased sophistication of seismic analyses—including both inertial analyses and analyses for liquefaction hazards. The result of these changes should be bridges that perform better and remain functional post-earthquake. These changes are expected to encourage alternatives to the traditional use of column plastic hinging, such as base-isolation. Over the last ten years, base-isolation has been used on few bridges in British Columbia—primarily retrofits of existing structures; however, given its ability to preserve post-earthquake functionality, base-isolation should be a serious consideration for any project.

1. Introduction – Evolution of Seismic Design Practice

Seismic design of bridges in British Columbia has evolved significantly in the last ten years, going from a bridge design code using outdated principles to a state of the art new code that implements performance-based design. The following timeline traces the key developments in this evolution using projects the authors were involved with.

- **2005 –** Bridges are designed to the *Canadian Highway Bridge Design Code* (*CHBDC*) *CAN/CSA-S6-00*). Key design features include a 475-year return period design earthquake, empirically defined response spectra, 1995 Geological Survey of Canada (GSC) seismic hazard maps, and force-based design with force modification factors (R-factors).
- **2005 –** Geological Survey of Canada (GSC) issues new seismic hazard maps for 100-, 475-, 975- and 2475-year return periods. The seismic hazard in Vancouver includes both a deterministic Mw8.2 Cascadia subduction interface event and probabilistic slab and crustal events.
- **2005 –** *National Building Code of Canada* (*NBCC*) shifts from a 475- to a 2475-year return period design earthquake and adopts the 2005 GSC hazard model with a uniform hazard response spectra (UHS).

- **2005 –** The BC Ministry of Transportation and Infrastructure (BC MoT) defines project-specific seismic design requirements for the Golden Ears Bridge, including 475-, 975- and 2475-year return period design earthquakes, UHS and explicit Performance Objectives.
- **2006 –** An updated *CHBDC*, *CAN/CSA-S6-06* is released with only minor revisions to the seismic design provisions. Both the 475-year return period earthquake and empirical response spectra are retained from *CAN/CSA-S6-00*, although the BC MoT *Supplement to CHBDC S6-06* (2007) ties the empirical response spectra to the PGA from the 2005 GSC seismic hazard maps. The updated code requires an evaluation "of the potential for liquefaction of foundation soils and the impact of liquefaction on bridge foundations".
- **2006 –** The Association of Professional Engineers and Geoscientists of British Columbia (APEGBC) releases initial guidelines for *Legislated Landslide Assessments for Proposed Residential Developments in BC* (2006) which addresses consequences for slope stability assessments and liquefaction effects with the *NBCC*'s shift from a 475-year to a 2475-year return period design earthquake.
- **2007 –** *Greater Vancouver Liquefaction Task Force Report* (Anderson et al. 2007) provides consensus design guidelines for buildings on liquefiable sites in accordance with *NBCC* 2005, but also generally relevant to liquefaction assessment for bridge design. Guidelines include specific magnitudes and PGAs for liquefaction assessment for both crustal and Cascadia subduction interface events.
- **2007 –** Project-specific seismic design requirements for the Coast Meridian Overpass in Port Coquitlam specify Performance Based Design for two levels: 475- and 2475-year return period earthquakes and adopt UHS. Project specifications also require separate analyses for liquefaction due to a Cascadia subduction interface event.
- **2008, 2009 –** Project-specific seismic design requirements for the Port Mann/Highway 1 Improvement Project and the South Fraser Perimeter Road specify performance based design for three levels, 475-, 975- and 2475-year return period earthquakes, as well as a Cascadia subduction interface event.
- **2010 –** GSC issues updated seismic hazard maps.
- **2012 –** Evergreen LRT: Project-specific seismic design requirements specify Performance Based Design for two levels: 100- and 975- year return period earthquakes; but with relaxed requirements for bridges and structures on liquefiable soils adjacent to Burrard Inlet. Project specifications also require two-dimensional nonlinear analyses for liquefaction hazards and coupled soil-structure interaction analyses at selected locations.
- **2014 –** An updated *CHBDC*, *CAN/CSA-S6-14* is released. Key features include multiple hazard levels, UHS from the upcoming 2015 GSC seismic hazard maps, and the performance-based design considering both service and damage outcomes at multiple ground motion levels. Liquefaction assessments will now be carried out for the 2475-year return period earthquake—similar to the *NBCC*'s transition in 2005 from 475 to 2475 years. The updated code also expands on CAN/CSA-S6-06 liquefaction requirements to include direction for staged assessment of liquefaction potential culminating in site response modeling.
- **2015 –** GSC will issue new seismic hazard maps that include a probabilistic ~Mw9.0 Cascadia subduction interface event along with slab and crustal events.

These developments comprise three major changes in seismic design practice: (1) improved understanding of seismic hazard—including raising the design earthquake from a 475-year to a 2475-year return period design earthquake with uniform hazard spectra and better knowledge of the contribution of the nearby Cascadia subduction zone; (2) a shift to a performance-based design philosophy with emphasis on improved post-earthquake performance (Fig. 1)—including multiple service and damage objectives for multiple levels of ground motions; and (3) increased sophistication of seismic analyses (Fig. 2)—including both inertial analyses and analyses for liquefaction hazards.

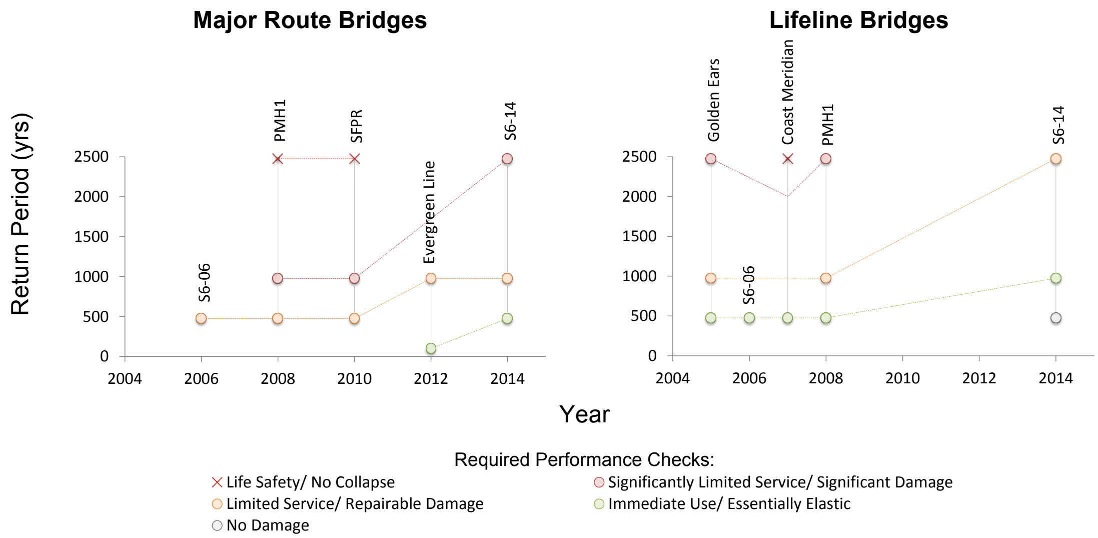

Fig. 1 – Evolution of Performance-Based Design Requirements

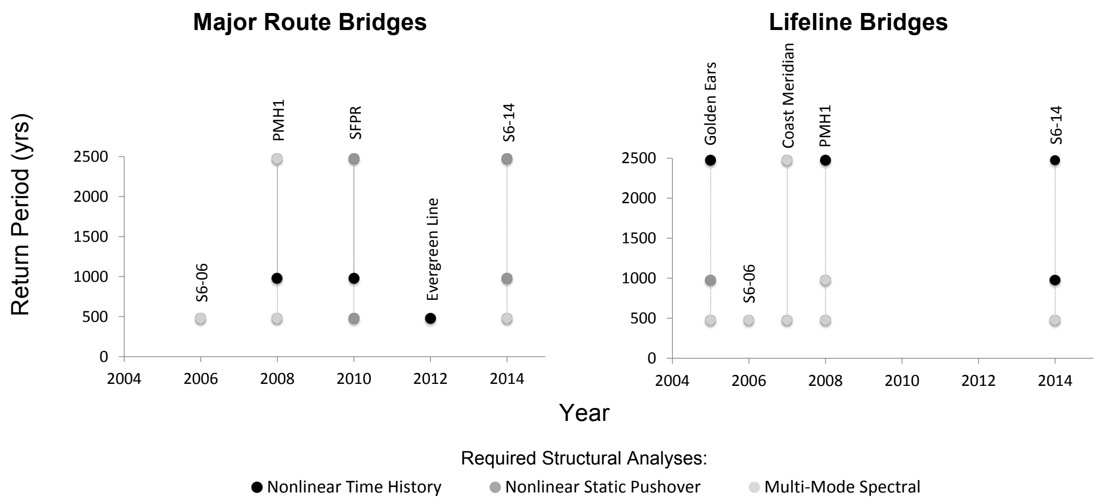

Fig. 2 – Evolution of Structural Analysis Requirements

2. Improved Understanding of Seismic Hazard

2.1. 2475-year Design Earthquake with Uniform Hazard Spectra

In 2005, the *National Building Code of Canada* adopted a 2475-year return period for the design earthquake, bringing it in line with US building codes. The change was too late, however, to be implemented in *CAN/CSA-S6-06*. Nonetheless, in British Columbia, the design earthquake level was raised to 2475 through project-specific seismic design requirements, first for the Golden Ears Bridge and subsequent major projects.

The change from 475- to 2475-year return period represents a five-fold increase in the design hazard level; or conversely, a five-fold reduction in the probability that the design accelerations will be exceeded. In B.C., the hazard curve is such that the 2475 spectral accelerations are approximately two times the 475 accelerations, as shown in Fig. 3.

At the same time as the GSC developed the 2475-year hazard, they also changed from the previous empirical spectra derived from the PGA to a uniform hazard spectra defined by spectral accelerations explicitly calculated at multiple periods. Unfortunately, *CAN/CSA-S6-06* still used the empirical spectra, which was first developed in 1981 for application in *ATC-6*, the first seismic design guidelines for bridges. The most notable difference between the two spectra is a significant reduction in spectral acceleration at longer periods (the empirical spectra artificially raised long period accelerations due to a lack of long period measurements at the time). The net effect is that the five-fold increase in design hazard is achieved with typically only a 30% increase in design accelerations, and much less at long periods.

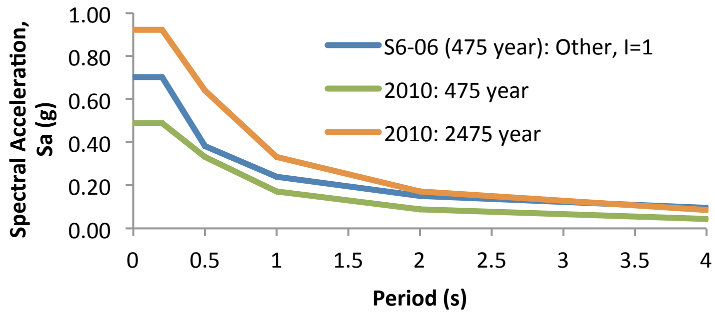

Fig. 3 – Comparison of 475-year and 2475-year response spectra for Vancouver

2.2. Contribution of the Cascadia Subduction Zone

Design for the Cascadia subduction interface event over the past 10 years was based on a deterministic Mw8.2 event in the 2005 GSC hazard model—even though this source was known to be capable of earthquakes Mw9.0 or larger. The deterministic event was a pragmatic selection based on data available at the time, as a dearth of usable ground motion records for large-magnitude earthquakes precluded the development of reliable ground motion prediction equations (GMPEs) for earthquakes larger than Mw8.5 (Adams and Halchuk 2003).

The subsequent Tohoku 2011 (Mw9.0) and Maule 2012 (Mw8.8) events provided the ground motion records necessary for development of reliable GMPEs for large-magnitude earthquakes (e.g. Stewart et al. 2013). The 2015 GSC hazard model incorporates these equations to provide a complete probabilistic model of the seismic hazard on the west coast, including the Cascadia subduction interface event. With this significant change in subduction hazard, the consensus understanding of the 2007 *Greater Vancouver Liquefaction Task Force Report* needs to be revisited, particularly with regards to the selection of appropriate magnitudes and PGAs for assessing liquefaction effects of the design earthquake.

3. Shift to Performance-Based Design

The change from single-level force-based design to multiple-level performance-based design represents a significant shift in seismic design philosophy: it is a recognition that designers can do better than simple collapse prevention. It recognizes the importance of bridges in post-earthquake response and recovery. It also reflects advances in our understanding of the seismic behaviour of bridges and the tools to control it. Rather than specify checking structural elements for specific actions—e.g., piles, columns, and girders for bending and shear—as per current prescriptive codes, performance-based design requires designers to take a whole-system approach that also includes non-structural components—e.g., joints and guide signs—that can affect the functionality of the bridge.

In the US, performance-based design has been in development for over 20 years. Much of the emphasis has been on establishing relationships between engineering design parameters (e.g., strains in concrete or reinforcement) and damage levels, and between damage levels and performance objectives. In 2003, *ATC-49* proposed a two-level performance-based design framework based around two performance objectives: functional and no-collapse. *ATC-49* was to be implemented as the next AASHTO specification, but did not get nationwide approval. Nevertheless, it has remained an influential guideline for implementation of performance-based design in Greater Vancouver, including the Golden Ears Bridge, Coast Meridian Overpass, Port Mann/Highway 1 Improvement Project and South Fraser Perimeter Road.

3.1. Performance-Based Design in British Columbia

3.1.1. Golden Ears Bridge

In Canada, GSC started calculating spectral accelerations for different hazard levels (100-, 475-, 975- and 2475-year return periods) with the 2005 hazard maps. This opened the door for design to multiple performance objectives using different hazard levels for each. In British Columbia, the first instance was the Golden Ears Bridge Project: for the main river crossing—designated as lifeline—BC MoT defined three levels of performance objectives and associated hazard:

- 475-year return period: Immediate Access / Minimal Damage
- 1000-year return period: Limited Access / Repairable Damage
- 2475-year return period: Significant Damage

Immediate Access/Minimal Damage was taken as an essentially elastic response, which is relatively easy to define and analyze. Significant Damage was taken as design to the Ultimate Limit State, using existing *CAN/CSA-S6-00* equations. For these first two, corresponding force modification factors (R-factors) could be selected for use with the current code provisions. Strain limits could also be derived from the code design principles. Although Limited Access/Repairable damage is vague and difficult to demonstrate; intermediate values of R or of strain limits could be used.

3.1.2. Coast Meridian Overpass

For the Coast Meridian Overpass Project, the City of Port Coquitlam considered the structure as Lifeline and defined two levels of design:

- 475 year return period: Functional (essentially elastic)
- 2475 year return period: Life-safety/no collapse (ULS design based on CAN/CSA-S6-06)

These objectives were selected because they did not require intermediate levels to be developed and were clearly defined from both the point of view of the owner and the design engineers.

3.1.3. Port Mann/Highway 1 Improvement Project/South Fraser Perimeter Road

With the Port Mann/Highway 1 Improvement Project (PMH1) and South Fraser Perimeter Road (SFPR) more intermediate performance objectives were defined (Table 1), moving away from the direct link with current code methodologies. Desired objectives were defined, but it was left to the design engineers to select engineering design parameters and associated limits to demonstrate the objectives were met.

Table 1 – PMH1/SFPR Seismic Performance Requirements

Classification	475-year + Subduction	975-year	2,475-year
Lifeline	Immediate Use	Limited access / Repairable damage	Possible loss of service/ Significant Damage but No Collapse
Economic Sustainability Route	Limited access / Repairable damage	Significantly Limited access / Significant Damage but No Collapse	Possible loss of service/ Significant Damage (Loss-of-span Prevention)

Limited access = 50% lanes, full access restorable within days after inspection
Significant Limited Access = emergency access, full access not expected until repairs

3.1.4. CAN/CSA-S6-14

The PMH1/SFPR approach led to the new *Canadian Highway Bridge Code*, *CAN/CSA-S6-14*. The code includes a full matrix of performance objectives, each defined as a combination of functional performance and damage level (Table 2).

Table 2 – CAN/CSA-S6-14 Seismic Performance Requirements

Classification	475-year	975-year	2,475-year
Lifeline	Immediate Service/ No Damage	Immediate Service/ Minimal Damage	Limited Service/ Repairable Damage
Major-Route	Immediate Service/ Minimal Damage	Limited Service/ Repairable Damage*	Service Disruption/ Extensive Damage
Other	Limited Service/ Repairable Damage	Service Disruption/ Extensive Damage*	Life Safety/Probable Replacement

* Optional requirement

Similar to PMH1/SFPR, these performance requirements were defined top-down: i.e., the desired end result is defined and it is up to the designer to determine the appropriate engineering design parameters and associated design limits—with the exception of some defined strain limits for concrete and reinforcement. Certain top-down objectives are not easily related to engineering design parameters—including access for emergency vehicles or restricted emergency vehicles, ground movements, and achieving specified time periods for returning to full function. This issue is discussed in Gérin and Onur (2010). Having different damage levels for the same functional objective also reduces clarity.

It is not clear if all the intermediate levels are necessary or practical—whether or not that is the case will become apparent as the new provisions are applied in practice. This is a new code with a significant change in design philosophy; therefore, it is certain there will be questions regarding its implementation. As it becomes more widely used and these questions are addressed, the code should converge towards a common application.

3.2. Performance-Based Design and Liquefaction

Ground response to liquefaction is nonlinear. Once accelerations are sufficiently large to trigger liquefaction at a site, ground movement due to lateral spread are more a function of the site gradient and the duration of the design earthquake than the magnitude of the accelerations. Because of this, the Coast Meridian Overpass, PMH1 and SFPR projects required separate consideration of the Cascadia subduction interface event requirements even though accelerations were lower than for other events. Since, once triggered, liquefaction hazards are most sensitive to these fixed site parameters, they can "step up" quickly from no ground displacement to significant ground displacement, with significant impacts to the bridge's post-earthquake performance. It is apparent that the nonlinear progression of liquefaction effects—and the corresponding cost of mitigating with ground improvement or structural accommodation—does not parallel the essentially linear progression of performance-based design requirements as a function of hazard level.

The economic implications of mitigating the liquefaction step change then becomes an important question for owners. In some cases it may be acceptable to reduce seismic performance requirements where liquefaction demands are expected to be onerous—as was the case with the Evergreen Line—rather than enforce the same seismic performance standard for liquefaction-induced ground movement and structural behaviour.

4. Increased Sophistication of Seismic Analyses

4.1. Structural Analyses

With multiple objectives and multiple design levels has come a corresponding increase in structural analysis requirements. This is particularly apparent in the PMH1 requirements that were reviewed in detail in Leggett and Gérin (2014). Similar to PMH1, the *CAN/CSA-S6-14* analysis requirements are a function of both the structure class (Lifeline, Major-Route, Other) and the return period (Table 3). They are based on the premise that there is a progression in importance or accuracy from multi-mode spectral to static pushover to nonlinear time history. Each analysis method, however, has a specific purpose and is used to measure different parameters.

Table 3 – CAN/CSA-S6-14 Seismic Analysis Requirements

Classification	475-year	975-year	2475-year
Lifeline	EDA	EDA+ ISPA+ NTHA	EDA+ ISPA+ NTHA
Major-Route	ESA (regular bridges) EDA (irregular)	EDA+ ISPA	EDA+ ISPA
Other	ESA (regular bridges) EDA (irregular)	ESA (regular bridges) EDA (irregular)	ESA (regular bridges) EDA (irregular)

ESA = uniform load method, single-mode spectral
EDA = multi-mode spectral, elastic time history (e.g. multi-support excitations)
ISPA = static pushover
NLTH = nonlinear time history

The current approach leads to some mismatches between performance objectives and analysis requirements: for example, for Other bridges at a 475-year return period, a static pushover analysis would typically be required to confirm whether repairable damage criteria are met. Thus, analysis requirements tied to the performance objectives to be demonstrated and to the type of structure might be more rational. The latter is important because different structures may be given different classifications. For example, in Quebec, major highway bridges are all designated Lifeline; thus there may be single-span bridges designated lifeline. Per *CAN/CSA-S6-14*, these would require 3 levels of analysis using all three methods; the same as a major cable-stayed bridge. Similar to implementation of performance-based design described above, these analysis issues will come to light and be rationalized as the new *CAN/CSA-S6-14* code is applied.

4.2. Analyses for Liquefaction Hazards

Advances in computing power have enabled increased use of sophisticated site response analyses using nonlinear effective stress constitutive models. In 2003, nonlinear finite element and finite difference analyses for liquefaction hazards were "normally beyond the scope of routine bridge design projects" (ATC 2003). By 2012, such analyses were a prescriptive requirement for design of the Evergreen Line. While sophisticated liquefaction hazard modeling provides a detailed understanding of liquefaction triggering and the site response mechanisms, its accuracy is still constrained by that of the underlying constitutive models, and the accuracy of those models is a subject of considerable debate.

Consider, then, how such results are applied to design. During the development of *ATC-49* investigators held a workshop to define ground displacement limits at Immediate Service and Significant Disruption performance levels. The participants concluded that "when uninterrupted or immediate service is desired, the permanent displacements should be small or nonexistent, and should be at levels that are within an accepted tolerance for normally operational highways of the type being considered"—i.e., on the order of tens of millimetres. Significant Disruption, on the other hand has recommended tolerances on the order of hundreds of millimetres—one or two orders of magnitude greater than those for Immediate Service.

CAN/CSA-S6-14 defines qualitative requirements for foundation performance at each service level/damage level, for which we have suggested nominal order-of-magnitude displacement limits (Table 4). As with *ATC-49*, *CAN/CSA-S6-14* features performance limits that span several orders of magnitude of ground displacement. *CAN/CSA-S6-14*, however, is more refined, featuring an intermediate, "Limited Service/ Repairable Damage" limit between minimal damage and extensive damage, and an additional "Life Safety/Probable Replacement" limit.

Table 4 – CAN/CSA-S6-14 Foundation Performance Criteria (after Table 4.16)

Service Level/ Damage Level (Table 4.16)	**Foundation Performance (Table 4.16)**	**Order-of-Magnitude Displacement**
Immediate Use/ Minimal Damage	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.	Millimetres to tens of millimetres
Limited Service/ Repairable Damage	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 repairs can bring the structure back to the original operational capacity.	Tens of millimetres to hundreds of millimetres
Service Disruption/ Extensive Damage	Ground lateral and vertical movements shall not exceed those that would prevent use by restricted emergency traffic after inspection or the bridge, nor preclude return of full service to the bridge.	Hundreds of millimetres
Life Safety/ Probable Replacement	Ground lateral and vertical movements are not restricted but shall not lead to collapse of the bridge superstructure.	Metres

The specified foundation performance limits are more refined than the expected accuracy of analyses for liquefaction hazard. It is possible to distinguish between the Immediate Service and Significant Disruption performance levels as per *ATC-49*, but it may not be possible to reliably distinguish the refined progression of the four service/damage levels in *CAN/CSA-S6-14*—particularly given variation in estimates for lateral spread can be as much as ±1 m (ATC 2003).

5. Potential Implications – Base-Isolation

The improved understanding of seismic hazard, shift to a performance-based design philosophy with emphasis on improved post-earthquake performance and increased sophistication of seismic analyses should result in bridges that perform better and remain functional post-earthquake. It's hoped that these changes encourage alternatives to the traditional use of column plastic hinging, such as base-isolation.

5.1. Background

The concept and efficacy of isolation as a means to reduce seismic vulnerabilities in structures is a relatively old one; however, its widespread adoption for new bridges, particularly short and medium span ones, has been slow to materialize in British Columbia. Most isolated bridges are in Italy, Japan, New Zealand and the United States. Following the philosophical switch from force-based design, focused largely on collapse prevention, to performance-based design, incorporating post disaster functionality into the objectives, should provide a significant incentive for the use of seismic isolation on more bridges.

Fundamentally, the primary benefit of isolation is due to a reduced structure stiffness and corresponding period shift, lowering the spectral acceleration and base shear a bridge experiences for a particular seismic event. In practice, it has been common to combine the period shift with increased damping, provided internal or external to the bearing, further reducing accelerations, shears and helping to minimize the increase in displacements. Seismic isolation, rather than ductile plastic hinges, can be used to absorb and dissipate seismic energy without significant damage to bridges. It has been suggested that bridges may be isolated at lower initial construction costs due to reduced foundation demands. Isolation provides the ability to achieve continued functionality even for large seismic events. Additional benefits of isolation versus ductility as a seismic design approach include more reliable energy absorption within manufactured isolation devices compared to crushing concrete and steel yielding, and the minimization or prevention of loss of service and associated revenues. (Chen, 2014)

Over the last 10 years in Greater Vancouver seismic isolation was used as a primary component of most of the seismic retrofits for the aging collection of major bridges including the Queensborough, Granville, and Burrard Bridges, among others. It has also been incorporated in new, long span bridges and their approaches such as the Port Mann Bridge and the Golden Ears Bridge. While the applicability of seismic isolation to large and strategic bridges remains, the vast majority of our local bridge network consists of less grandiose small and medium span bridges. Considering the significant number of these 'typical' bridges that have been built locally over the last 10 years the comparatively low number that are isolated is surprising and provides opportunity for improvement.

5.2. Potential Factors Limiting Recent Use

A number of potential factors have likely contributed to the limited use of seismic isolation on typical bridges in BC. Without backing of an exhaustive study quantifying the impact of the various factors, the following serves as but one view on what may be involved.

From a philosophical perspective, the industry has slowly been transitioning from a force-based environment, focused solely on collapse prevention, to a performance-based one incorporating structural, functional and service criteria for a handful of seismic hazards. The governing codes and, to a varying but generally lesser extent, project-specific criteria have provided limited incentive, or justification, for bridge engineers to exceed the status quo. There has always been considerable pressure to design bridge projects to satisfy minimum code requirements for the lowest initial costs.

Many recent bridge projects has been delivered under some form of a design-build environment, having the tendency to ensure design solutions focus on minimizing upfront construction costs as the paramount goal within the limits of the contractual criteria. Adoption of innovative designs generally requires them to be cheaper and/or faster than the acceptable alternatives, with the onus for making the argument often landing with the engineer. The fast-tracked nature of these projects further limits the ability to pursue potentially more time consuming, but preferential, solutions. Whether merit exists or not, the perception has been that seismic isolation is more costly due to the increased complication with bearings and their testing requirements.

From a technical nature, it is plausible that local engineers have been slower to embrace seismic isolation due to their lack of familiarity with the analysis and design of these systems. Furthermore, many of the local bridges are founded on soft soils and have relatively flexible foundations, leading to fewer short period structures and minimizing the benefits of a period shift. Combined with the lack of increased damping for cost efficient bearings, the benefit of isolation becomes less tangible in these cases. Control of deflections, both service and ultimate, may also create additional complications.

Seismic isolation has generally been achieved by using relatively sophisticated proprietary bearing products, while there are benefits to these systems and depending on the design parameters, including loads, displacements and energy absorption requirements, they may be necessary for many bridges. However, for smaller bridges, typical laminated elastomeric bearings have been used for isolation purposes, sufficient for service and ultimate demands in many cases.

5.3. Potential Impact of *CAN/CSA-S6-14*

The question has become do the recent changes in seismic design practice, including the requirements of *CAN/CSA-S6-14*, provide sufficient incentive on their own to sway the industry to use seismic isolation on a more regular basis. Current AASHTO guidelines recognize the benefits and promote the use of seismic isolation for new bridges (Chen, 2014).

In *CAN/CSA-S6-14* the use of seismic base isolation generally has lower minimum analysis requirements compared to similar structures relying on ductility-based approaches. Isolated bridges with periods below 3 seconds and damping below 30%, capturing the majority of typical structures, do not require time history analysis. The essentially elastic response of substructures negates the need for inelastic analysis to demonstrate performance criteria have been met, reducing the overall effort of performance-based design. It should also be more likely that an isolated bridge will significantly exceed the performance-based objectives of *CAN/CSA-S6-14*.

Not surprisingly, bearing testing requirements remain much more involved for bearings being used for isolation. However, the associated costs should reduce as their use increases. Proprietary bearing suppliers are now established, have performed prototype tests on a range of bearing sizes, and are familiar with typical quality control testing requirements.

6. Conclusions

Seismic design of bridges in British Columbia has evolved significantly in the last ten years. Developments have comprised three major changes in seismic design practice: (1) improved understanding of seismic hazard—including raising the design earthquake from a 475-year return period to 2475-year return period and better knowledge of the contribution of the nearby Cascadia subduction zone; (2) a shift to a performance-based design philosophy with emphasis on improved post-earthquake performance—including multiple service and damage objectives for multiple levels of ground motions; and (3) increased sophistication of seismic analyses—including both inertial analyses and analyses for liquefaction hazard. The result of these changes should be bridges that perform better and remain functional post-earthquake.

Initially implemented in British Columbia through project-specific requirements, Performance Based Design is now an integral part of the new *CAN/CSA-S6-14* bridge design code. Given the significant change this represents in how designers approach seismic design, coupled with this being the first bridge code to fully implement PBD, there is bound to be questions regarding the performance objectives and how they are met. As the new code is applied in practice, these questions will be addressed and the code requirements will be interpreted – the result should be an evolution of the practice where highway bridges perform better and remain more functional after earthquakes than under the previous code.

The new code provisions are expected to encourage alternate seismic design strategies such as base-isolation. Over the last ten years, base-isolation has been used on few bridges in British Columbia—primarily retrofits of existing structures; however, given its ability to preserve post-earthquake functionality, base-isolation should be a serious consideration for any project.

7. References

ADAMS, John & HALCHUK, Stephen, “Fourth generation seismic hazard maps of Canada: Values for over 650 Canadian localities intended for the 2005 National Building Code of Canada”, *Open File 4459*, Geological Survey of Canada, Ottawa ON, 2003.

ANDERSON, D.L. et al., “Geotechnical Design Guidelines for Buildings on Liquefiable Sites in Accordance with NBC 2005 for Greater Vancouver Region”, Greater Vancouver Liquefaction Task Force, Vancouver BC, May 2007.

APEGBC, “Guidelines for Legislated Landslide Assessments for Proposed Residential Developments in BC”, Association of Professional Engineers and Geoscientists of BC, Burnaby BC, 2006.

ATC, “Seismic Design Guidelines for Highway Bridges”, *ATC-6*, Applied Technology Council, Berkeley CA, 2003.

ATC/MCEER, “Recommended LRFD Guidelines for the Seismic Design of Highway Bridges”, *ATC-49*, ATC/MCEER Joint Venture, Redwood City CA, 2003.

BC MoT, “Bridge Standards and Procedures Manual”, *Volume 1—Supplement to CHBDC S6-06*, British Columbia Ministry of Transportation, Victoria BC, August 2007.

CHEN, Wai-Fah &DUAN, Lian, “Bridge Engineering Handbook – Second Edition – Seismic Design”, CRC Press, Boca Raton FL, 2014.

CSA, “Canadian Highway Bridge Design Code”, *CAN/CSA-S6-00*, Canadian Standards Association, Mississauga, ON, 2000.

CSA, “Canadian Highway Bridge Design Code”, *CAN/CSA-S6-06*, Canadian Standards Association, Mississauga, ON, 2006.

CSA, “Canadian Highway Bridge Design Code”, *CAN/CSA-S6-14*, Canadian Standards Association, Mississauga, ON, 2014.

GÉRIN, Marc & ONUR, Tuna, “A Review of Performance Requirements for Bridges, 9th U.S. National & 10th Canadian Conference on Earthquake Engineering”, EERI, Toronto ON, 2010, 10 p—paper in CD-ROM proceedings.

LEGGETT, Casey & GÉRIN, Marc, “Port Mann / Highway 1 Improvement Project: Performance Based Seismic Design”, *9th International Conference on Short and Medium Span Bridges*, Canadian Society for Civil Engineering, Calgary AB, July 2014, 10 p—paper in conference proceedings.

NRCC, “National Building Code of Canada”, National Research Council Canada, Ottawa, 2005.

STEWART, Jonathan et al., "Implications of the Mw 9.0 Tohoku-Oki Earthquake for Ground Motion Scaling with Source, Path, and Site Parameters", *Earthquake Spectra*, Volume 29, No. S1, March 2013, pp S1–S21.