CSA-S6-19 Section 4 - the 2nd generation of Performance Based Seismic Design Provisions

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ABSTRACT

Performance Based Design (PBD) was first introduced for the seismic design of bridges in the CSA-S6-14 edition of the Canadian Highway Bridge Design Code. CSA-S6-19 includes a number of changes to improve clarity and better achieve the objectives of performance-based seismic design. In particular, the performance levels applicable to bridges in each importance category have been reduced from three to two, mainly by removing optional levels. Damage indicators have been revised following additional research conducted since the previous version that found some criteria unduly conservative in certain conditions. In the new version, capacity-design principles are clarified and better integrated with performance-based requirements, and the foundations section is significantly expanded to fully implement performance-based principles in the design of geotechnical systems. This includes the definition of new performance criteria specifically for geotechnical systems, both within and beyond the bridge embankment zone. And finally, more extensive guidance is provided for existing bridges, while still allowing sufficient flexibility for owners to tailor requirements to the particular conditions encountered with existing bridges.

Keywords: S6-19, Bridges, Performance-based design, Capacity-based design, Existing bridges

INTRODUCTION

The 2019 version of the Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-19 (S6-19) [1], includes the 2nd generation of Performance Based seismic design provisions. The previous version, CAN/CSA-S6-14 (S6-14) [2] was the first bridge code to implement Performance-Based Design (PBD) as an explicit requirement for seismic design and represented a significant change in seismic design philosophy prescribed by the code. S6-19 includes a number of changes to improve clarity and better achieve the objectives of performance based seismic design. Key changes include revisions to the performance levels applicable to bridges in each importance category and revisions to the performance criteria defining certain damage levels. Feedback from practitioners, academics and owners on the original provisions was also incorporated in the new provisions.

PERFORMANCE LEVELS

The Performance Levels are the foundation of the performance-based seismic design approach used in the CHBDC. They define service and damage performance objectives as a function of the bridge importance category: lifeline, major-route, or other bridges. In S6-19, the definition of lifeline bridges no longer refers to the type of structure (S6-14 referred to a “large, unique, iconic, and/or complex structure that represents significant investment and would be time-consuming to repair or replace”) and instead only focuses on the importance of the structure (“a bridge that is vital to the integrity of the regional transportation network and the continuous function of the regional or wider economy, or to the security of the region”). This brings the definition of lifeline bridges in line with the other two importance categories and clarifies the relationship between importance categories and performance levels. This will make it easier for owners to categorize their bridges in terms of desired performance and bring more uniformity in the application of the PBD provisions.

In S6-14, there were three performance levels defined for each importance category - one for each of the three ground motions considered in the code, 10%, 5% and 2% probability of exceedance in 50 years (475 years, 975 years, 2475 years return periods). In S6-19, the number of performance levels is reduced and levels that were marked as “optional” were eliminated to improve clarity and uniformity of application. The revised performance levels defined in S6-19 are shown in Table 1.

Lifeline bridges included “none” and “minimal” damage levels for 10% and 5% ground motions respectively, but they were both associated with the same “immediate” service level. Because of this, there is no practical difference between the two levels and the no damage for the 10% ground motion criteria is expected to be met when the minimal damage criteria are met for the 5% ground motions. Thus the 10% ground motion level was eliminated for lifeline bridges. This eliminates the ambiguity of having two different damage levels associated with the same service level.

For Major-route bridges, the 5% ground motion level has been eliminated. It was previously marked as “Optional unless required by the Regulatory Authority or the Owner” as it is bounded by the 10% and the 2% ground motion levels. Research by Khan and Gerin [3] confirmed that this level was unlikely to govern the design of major-route bridges. Also, having optional levels created ambiguity and had little support among Owners. The table defines minimum levels; thus, Owners can still add additional levels within the context of the code.

The “Other bridges” category is aimed at bridges that are not on emergency response routes, where alternate routes are readily available, that have low use, or are not on public roads – in other words, bridges where service is not needed post-earthquake. For these bridges the primary design objective is life-safety at the 2% ground motion level. In S6-14, optional levels were defined for the 10% and 5% ground motion levels: for S6-19, the 5% level was removed and the 10% level was made mandatory to eliminate ambiguity with the minimum requirements. The Life Safety/Probable Replacement performance level is on par with what is expected from a force-based design, which makes this requirement consistent with the code allowing regular “other” bridges and irregular “other” bridges in Seismic Performance Category 2 to be designed using Force Based Design.

Table 1. Minimum Performance Levels.

Seismic ground motion probability of exceedance in 50 years (return period)	**Lifeline bridges**		**Major-route bridges**		**Other bridges**	
	Service	**Damage**	**Service**	**Damage**	**Service**	**Damage**
10% (475 years)	–	–	Immediate	Minimal	Service Limited	Repairable
5% (975 years)	Immediate	Minimal	–	–	–	–
2% (2475 years)	Service Limited	Repairable	Service Disruption	Extensive	Life Safety	Probable Replacement

The changes to the minimum performance levels provide a clearer differentiation between bridge importance categories and between damage levels. They also remove ambiguity caused by having optional levels in the definition of minimum requirements and align the performance levels more closely with the criteria for when PBD is required or FBD is permitted. The result is a two-level design that focuses on the most important levels and eliminates unnecessary levels.

DAMAGE INDICATORS FOR CONCRETE SUBSTRUCTURE ELEMENTS

The damage indicators are the engineering parameters that are used to demonstrate the design satisfies the damage levels specified above. The damage indicators for concrete substructure elements were modified in S6-19 compared to S6-14 (Table 2). The most significant change is the increase in allowable steel strain, ε_s, from yield (typically 0.002) to 0.01. The previous criterion was found to be too conservative in some locations within Canada, particularly for bridges on soft soils in British Columbia. Strain limits at the extreme fibers of circular columns in these cases in particular were found challenging.

Minimal Damage

For concrete elements the concrete compressive strains for minimal damage must not exceed 0.006. This strain limit corresponds to limited concrete cover spalling. Initial cover spalling has been reported as occurring at compressive strains varying from 0.004 (Priestley et al. [4]) to an average value from 10 columns subjected to reversed cyclic loading of 0.006 (Lehman et al. [5]). It is noted that initial spalling in circular columns occurs in only a limited zone and spalling in general occurs above the base of the column due to the fact that the cover is restrained by the foundation. For concrete elements the vertical reinforcing bars are permitted to experience yielding but with a stain limit of 0.010.

Table 2. S6-14 and S6-19 damage indicators for concrete substructures.

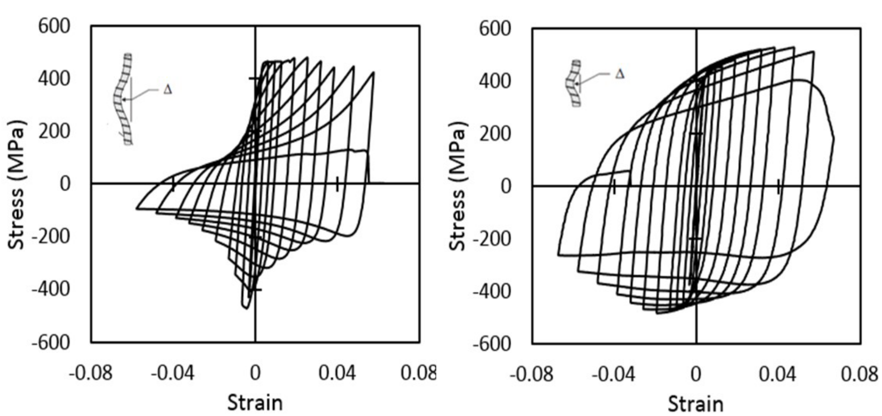
	S6-14	**S6-19**
Minimal Damage	$\varepsilon_c \leq 0.004$ $\varepsilon_s \leq$ yield	$\varepsilon_c \leq 0.006$ $\varepsilon_s \leq 0.01$
Repairable Damage	$\varepsilon_s \leq 0.015$	$\varepsilon_s \leq 0.025$
Extensive Damage	ε_c: concrete core shall not crush $\varepsilon_s \leq 0.05$	ε_c: core concrete shall not exceed 80% of ultimate strain $\varepsilon_s \leq 0.05$

Repairable Damage

For reinforced concrete elements the strains in the vertical reinforcing bars are limited to 0.025. This limits strain hardening of the steel reinforcement.

Extensive Damage

While extensive cover spalling is permitted, the strain in the concrete in the confined core is limited to 80% of the ultimate confined concrete strain limit. At this damage state the tensile strain in the vertical reinforcing bars is limited to 0.05. This strain limit is intended to avoid significant buckling of the vertical bars between the spiral reinforcement or the hoops and crossties. It is considered that buckling of the bars in compression occurs after significant tensile straining occurs and then upon load reversal the bars experience buckling. Figure 1 shows the influence of the spacing of the transverse reinforcing steel on the reversed cyclic loading of bare bars. For the case of a spacing of transverse reinforcement equal to 12 times the longitudinal bar diameter (Fig. 1(a)), bucking and degradation of the bar's compressive response occurs after reaching a tensile strain of about 0.01. The CHBDC limits the spacing of transverse reinforcement to six vertical bar diameters. Figure 1(b) shows the much-improved response of the vertical bars when this limit is applied, with little degradation in the compressive response of the vertical bars after reaching a tensile strain exceeding 0.05.



(a) (b)

Figure 1. Reversed cyclic loading of Grade 400W 20M reinforcing bars with transverse reinforcement spacing of (a) 12 bar diameters and (b) 6 bar diameters (Adomat [6] and Howard [7]).

Probable Replacement

Severe damage is likely but crushing of the confined concrete core must be avoided. The reinforcing bar tensile strains shall not exceed 0.075, except that for steel reinforcing of 35M or larger the strains shall not exceed 0.060.

CAPACITY DESIGN PRINCIPLES AND DESIGN CONSIDERATIONS FOR CONCRETE COLUMNS

Capacity Design

When PBD was added in S6-14 it created some ambiguity regarding the application of capacity design principles. Some of the clauses in S6-19 have been modified to more fully integrate PBD and clarify that capacity design is an essential feature in obtaining a ductile bridge structure, whether PBD or FBD is used. Capacity design is accomplished by clearly identifying the ductile elements that are designed and detailed to undergo inelastic hinging and the capacity-protected elements that are designed to remain elastic.

Expected Nominal Resistances

The CHBDC uses "expected nominal resistances" for the design of the ductile substructure elements. The expected nominal resistance of a ductile element is determined as the nominal flexural resistance, assuming material resistance factors for concrete and reinforcing bars of 1.0, and assuming expected material properties.

The expected yield strength of reinforcing bars is taken as the minimum specified yield strength times a factor of 1.1 for ductile substructure elements in low-ductility systems or a factor of 1.2 for ductile substructure elements in high-ductility systems. These factors take account of the ability of more ductile elements to develop higher stresses and to account for the difference between the actual yield stress and the minimum specified yield stress. The expected compressive strength of concrete is taken as 1.25 times the specified compressive strength of concrete.

Required Factored Shear Resistance

The seismic design forces for capacity-protected elements must have factored resistances equal to or greater than the maximum force effect that can be developed by the ductile substructure element(s) attaining their probable resistances. For yielding mechanisms involving flexural hinging in ductile concrete substructure elements such as columns, piers, and bents, inelastic hinging moments are taken as their probable resistance determined by multiplying the flexural expected nominal resistance of concrete sections by 1.30 (Fig. 2). A similar approach is taken for steel ductile substructure elements. Capacity design principles are also used to avoid brittle failure mechanisms such as shear failures in reinforced concrete columns. The design shear force is determined from static plastic analysis considering the probable flexural resistance of the member and its effective height.

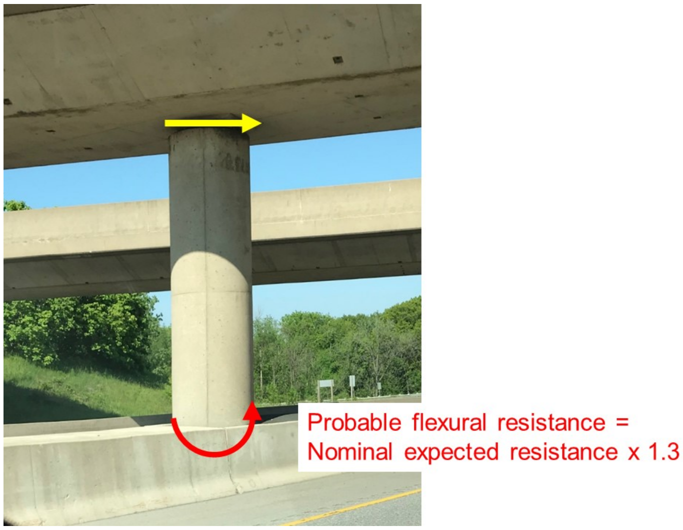


Figure 2. Determining design shear force corresponding to flexural hinging.

The shear reinforcement is designed in accordance with the requirements of the Simplified Method for shear design (Clause 8.9.3) with β taken as 0.18 and θ taken as $42°$. This change implemented in the S6-19 gives an increase in the shear resistance from the S6-14 provisions. This was deemed appropriate given the significant amount of vertical reinforcement and confinement reinforcement in typical columns. The transverse reinforcement consists of hoops, seismic crossties or spirals. In lieu of this simplified method the general method for shear design (Clause 8.9.3.7) may be used. Alternatively, the method used by CALTRANS [8] that is based on the research of Kowalsky and Priestley [9] and Priestley et al. [10], may be used.

Additional Considerations for Columns

For flared columns and columns attached to partial height walls, the top and bottom flares and the height of the walls shall be considered in determining the effective column height. The presence of a flare in a column can significantly reduce the column height and hence increases the shear (see Fig. 3(a)). In the region near the bottom of an extended pile bent the plastic hinge region shall be considered to extend from a low point of three times the maximum cross-section dimension below the calculated point of maximum moment, accounting for soil-pile interaction (see Fig. 3(b), to an upper point at a distance of not less than the maximum cross-section dimension, and not less than 500 mm, above the ground line.

Figure 3. (a) Shear failure of a column with a flare in Northridge earthquake (Mitchell et al. [11]) and (b) column in pile bent embedded in soft soil showing soil-structure interaction (Mitchell et al. [12]).

FOUNDATION DESIGN

For S6-19, the requirements pertaining to seismic design of foundations (Section 4.6 in S6-14) have been moved to Section 6 Foundations and Geotechnical Systems in order to collate all foundation provisions in one Chapter. The requirements have also been significantly expanded to more fully implement PBD for foundations and geotechnical systems.

Even though the foundation requirements have been removed from Section 4, the structural design of bridge foundations, abutments, retaining walls and geotechnical systems still needs to consider any relevant soil-structure interaction in the seismic analyses (e.g., through the use of linear or nonlinear soil springs) in order to capture the effects of the soil on structural stiffness, identify soil forces applied to the structure or any limitations or changes to the displacement characteristics of the structure.

For the first implementation of PBD for foundations, S6-14 referenced the static geotechnical resistance factors for use when targeting an essentially-elastic performance and specified a resistance factor of 1.0 when targeting life-safety performance or when verifying capacity-protected elements. In S6-19, a geotechnical resistance factor of 1.0 is specified for all performance objectives. This is a significant change in design philosophy: for the performance-based seismic design of foundations, the factored-strength force-based approach – still valid for static loads – is now fully replaced by a displacement-based approach where the non-linear and post-peak behaviour of the soil and geotechnical systems can be utilized, provided the specified performance objectives are met. This provides better integration of soil-structure interaction within the framework of the code and better support for innovative solutions.

The foundation section now provides more detailed requirements and guidance specific to shallow foundations, deep foundations and abutments regarding soil-structure interaction and performance-based design requirements. Section 6 defines performance criteria specifically for foundations and geotechnical systems: Table 3 summarizes the performance criteria for geotechnical systems within the approach embankment zones of a bridge; other criteria (not shown) are also defined for geotechnical systems outside the embankment zone. Other changes include a more extensive consideration of liquefaction and its impact on performance and explicit requirements for combining inertial and kinematic effects.

The new provisions fully implement PBD for the seismic design of foundations and geotechnical systems and are more integrated than before with the bridge importance categories and the structural performance levels. The new provisions also require closer cooperation between Geotechnical and Structural engineers to determine acceptable deformations in the foundations, to assess the impact of ground deformations on the structure and, ultimately, to demonstrate performance levels are met.

Table 3. Performance criteria for geotechnical systems.

Seismic ground motion probability of exceedance in 50 years (return period)	**Lifeline geotechnical systems**	**Major-route geotechnical systems**	**Other geotechnical systems**
10% (475 years)	–	100% of the travelled lanes available	50% of the travelled lanes, but not less than one, available
5% (975 years)	100% of the travelled lanes available	–	–
2% (2475 years)	50% of the travelled lanes, but not less than one, available		not collapse

EXISTING BRIDGES

The seismic assessment and retrofit design of existing important bridges in British Columbia have employed displacement-based seismic design methods since the early 1990's [13, 14, 15]. Several major river and harbour crossings in the Lower Mainland of B.C. were upgraded in the two decades since using displacement-based design methods. These projects typically provided a 'safety' level (collapse prevention) seismic upgrading, in some cases also requiring use of the crossing by emergency traffic for post-earthquake response, to a 10% in 50-year hazard level. The use of displacement-based methods was believed to be essential for the seismic performance assessment and design of retrofits since these bridges had poor, brittle details and little or no seismic resilience. The use of reduction factors as a surrogate measure of ductility demand or capacity, as was common for new bridges, was considered inadequate for these existing bridges. As such, these projects laid the foundation for the adoption of performance-based design in new bridges in BC, and in turn to the formal implementation of PBD in the Canadian Bridge code in 2014.

As stewards of transportation infrastructure in Canada, Owners and their engineering advisors are required to make informed, far-reaching decisions regarding bridge and transportation network management policy, ensuring critical network availability for post-seismic response and recovery, and renewal or replacement of existing bridges for functional, risk, reliability, condition and seismic reasons. S6-14 provided significant leeway to Owners in these areas for existing bridges. Initially, bridge engineers and owners composed project-specific seismic criteria documents for retrofit projects. The Province of BC published their best practices for bridge retrofit [16] and updated it [17] to supplement S6-14. During the past decade in particular, Owner's seismic performance objectives for existing bridges in BC and elsewhere have increased. Currently, BC targets performance levels for existing bridges [18] that essentially matches the minimum requirements for new bridges within S6-14. In part, this has been economically achievable on several recent projects, however, these bridges may not have included the challenges of loose, weak or liquefiable soils, or even the higher seismic hazard levels of other areas in southwestern BC. In addition, the Province is expected to revisit seismic hazard levels and performance in the context of updates to the bridge code within S6-19. Bridge owners across Canada have sought additional guidance in the application of PBD to existing bridges within S6-19. As a result, recommended performance objectives for existing bridges was included within the public review draft of S6-19, and in turn included in the commentary document, S6.1-19 [19], that provides guidance and background to the application of the formal code. Table 4 below outlines these recommended minimum levels for existing bridges.

Table 4. Recommended Minimum Performance Levels for Existing Bridges.

Seismic ground motion probability of exceedance in 50 years (return period)	**Lifeline bridges**		**Major-route bridges**		**Other bridges**	
	Service	**Damage**	**Service**	**Damage**	**Service**	**Damage**
10% (475 years)	–	–	–	–	–	–
5% (975 years)	–	–	Life Safety	Probable Replacement	Life Safety	Probable Replacement
2% (2475 years)	Life Safety	Probable Replacement	–	–	–	–

While not legally mandated in Canada, these recommended minimum levels reflect a recognition of the critical importance of important bridges to the post-earthquake recovery and response in Canadian society. For Lifeline and Major Route bridges, it is anticipated that Owners may adopt a higher standard that allows for a rapid return to service for emergency traffic and thereafter for use by the public. In practice this has been difficult to achieve for some bridges, owing either to being founded on liquefiable soils prone to lateral spreading, or to the technical, economic or aesthetic challenges of providing reliability and resilience to older structures. It is expected that seismic retrofit using isolation or other low-damage systems will become increasingly common as Owners target a return to service of critical bridges [20].

Neither S6-19 nor S6.1-19 contain damage or strain descriptions that are applicable to existing bridges. Bridges in Canada constructed with deficient and brittle details in concrete or steel sub-structures will continue to require project-specific damage states to be defined for the use of PBD. It is recognized that strain limits at specific, isolated locations within a bridge, such as at the extreme fibre of the first assessed plastic hinge location within a column of one bridge pier, may lead to a conservative bridge system overall from a performance-based perspective for new, well-detailed bridges. However, for existing bridges that are susceptible to early, brittle failure modes in gravity-load carrying components, loss of concrete cover in the first or any hinge can potentially be catastrophic. Damage states and performance assessments shall take such conditions into account in the retrofit of existing bridges.

Another aspect of the retrofit of existing bridges, as opposed to the design of new bridges, is that seismic retrofit is commonly done as part of a broader upgrade of a bridge to extend its service life or to improve it functionally. Because they are also carrying existing traffic, or owing to funding constraints in a given fiscal year, upgrading works are sometimes staged in multiple contracts. The preferred packaging and staging can significantly impact decisions on seismic upgrades and timing. Or, upgrading works may be urgently required, prior to completing a full seismic assessment or retrofit design. These factors all affect an Owner's approach, and each project will have unique and unpredictable challenges. As such, seismic upgrading objectives have been expressed as a recommended minimum framework within the commentary to the code to allow Owners to achieve societal or governmental objectives in an effective manner.

CONCLUSIONS

The 2019 version of the Canadian Highway Bridge Design Code, S6-19, includes the 2nd generation of Performance Based seismic design provisions. Performance levels reduced to two levels per importance category in order to focus on the more important performance objectives and eliminate the ambiguity associated with optional levels. Damage indicators have been adjusted to remove some of the conservatism observed with the first generation of requirements. The requirements for capacity design have been clarified and better integrated with the requirements for performance-based design. The foundations section has been significantly expanded to fully implement a performance-based approach for the seismic design of foundations and geotechnical systems. Performance-based design has also been given a framework for the assessment and retrofit of existing bridges within S6-19 and the commentary, S6.1-19. All these changes result in a clearer, more integrated application of performance-based seismic design within the framework of the bridge code.

Having been applied formally across Canada since S6-14, and for two decades in some Provinces, it is hoped that Performance Based Design will increasingly open the door to more resilient bridges in Canada, with a wider range of demonstrable, innovative and low-damage seismic load-resisting systems.

REFERENCES

[1] Canadian Standard Association - CSA (2019). CAN/CSA-S6-19: Canadian Highway Bridge Design Code.
[2] Canadian Standard Association - CSA (2014). CAN/CSA-S6-14: Canadian Highway Bridge Design Code.
[3] Khan, S. and Gerin, M. (2018). Application and Comparison of CSA-S6 Performance-Based Seismic Design Provisions, Proceedings of the 10th International Conference on Short and Medium Span Bridges, Quebec City, July 31 – August 3, 2018.
[4] Priestley, M.J.N., Seible, F., and Calvi, G. M. 1996. "Seismic Design and Retrofit of Bridges", Wiley, New York.
[5] Lehman, D. E., Moehle, J. P. 2000 "Seismic Performance of Well-Confined Concrete Bridge Columns", PEER 1998/01, Pacific Earthquake Engineering Research Centre, Berkeley, CA.
[6] Adomat, N. (2017). "Effect of hoop spacing on the response of coupling beams', M.Eng. thesis, Dept. of Civil Engineering, McGill University.

[7] Howard, B. (2017). "Seismic response of diagonally reinforced coupling beams with varied hoop spacings"', M.Eng. thesis, Dept. of Civil Engineering, McGill University.
[8] Caltrans. 2013. Seismic Design Criteria, Version 1.7, April (2013).
[9] Kowalsky, M.J. and Priestley, M.J.N. (2000). "Improved Analytical Model for Shear Strength of Circular Columns in Seismic Regions", ACI Structural Journal V97, S42, pp. 388-396.
[10] Priestley, M.J.N., Seible, F., and Calvi, G.M. (1996). Seismic Design and Retrofit of Bridges. John Wiley and Sons, New York.
[11] Mitchell, D., Bruneau, M., Williams, M., Anderson, D.L. and Sexsmith, R.G. (1995). "Performance of Bridges in the 1994 Northridge Earthquake", CJCE, V22, N2, pp. 415-427.
[12] Mitchell, D., Tinawi, R. and Sexsmith, R.G. (1991). "Performance of Bridges in the 1989 Loma Prieta Earthquake - Canadian Design Concerns", CJCE, V18, N4, pp. 711-734.
[13] Kennedy, D., Turkington, D. 1991. *Seismic Assessment and Retrofit of the Vancouver Oak Street Bridge*, Pacific Conference on Earthquake Engineering, Auckland, New Zealand, November 1991
[14] Kennedy, D., 2003. Mission Bridge - Displacement - Based Seismic Retrofit Design, 2003 ASCE Structures Congress, Seattle, Washington, U.S.A.
[15] Priestley, M.J.N., Seible, F., and Chai, 1992. Design Guidelines for Assessment, Retrofit and Repair of Bridges for Seismic Performance. Report No. SSRP-92/01, University of San Diego.
[16] Ministry of Transportation of British Columbia, 2005. Seismic Retrofit Design Criteria, June.
[17] Ministry of Transportation and Infrastructure of British Columbia, 2016. BC Supplement to CSA S6-14.
[18] Performance-Based Seismic Design of Bridges in BC, 2018. Engineers and Geoscientists of BC, Volume 1.0, March.
[19] Canadian Standard Association - CSA (2019). S6.1-19: Commentary to the Canadian Highway Bridge Design Code.
[20] Kennedy, D., Dowling, J., Du, H., 2017. Opportunities in the Performance Based Seismic Design of Bridges in British Columbia, 39th IABSE Symposium, Vancouver, B.C., Canada, September.