Performance-based Design (PBD) of British Columbia Bridges

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SUMMARY
Bridges built in British Columbia (BC) for the BC Ministry of Transportation (MoT) generally are designed following the Canadian Highway Bridge Design Code (CAN/CSA S6-06), the MoT Bridge Standards and Procedures Manual and project specifications that include some PBD provisions. It is intended that for important bridges a new PBD design code would be introduced covering the entire design process. For the owner or steward of structures, it is important to understand what structural and functional performance they can expect, rather than just that the structure design met a code. Performance-based seismic design (PBD) requires that the structure should meet certain performance criteria at specified levels of seismic hazard or probability of occurrence. The present paper provides a review of a comprehensive standalone PBD code that is being proposed for the MoT which is intended to provide a functional performance guideline.

Keywords: Performance-based, design, code, bridge

1. FOREWORD

Bridges built in BC for the BC Ministry of Transportation (MoT) generally are designed following the Canadian Highway Bridge Design Code (CAN/CSA S6-06), the MoT Bridge Standards and Procedures Manual and project specifications that include some PBD provisions. It is intended that for important bridges a new PBD design code would be introduced covering the entire design process. The present paper provides a review of a comprehensive standalone PBD code that is being proposed for the MoT which is intended to provide a functional performance guideline.

For the owner or steward of structures, it is important to understand what structural and functional performance they can expect, rather than just that the structure design met a code. For the MoT, knowing what performance can be relied on and what damages may occur allows it to directly relate the prioritization of funding to the importance of the structures and routes. Also retrofit strategies and post-disaster planning can better incorporate risk assessment and resourcing.

Performance-based seismic design (PBD) requires that the structure should meet certain performance criteria at specified levels of seismic hazard or probability of occurrence. The performance criteria may simply state that there should be no collapse at a low probability of seismic occurrence, and at higher levels of seismic probability or for different bridge classification it may be more prescriptive in defining limiting deformations or damage to members. The performance criteria do not directly require a certain strength level (R factor in North America). However limiting damage may require limiting deformation (if it cannot be accommodated) which usually means changing the stiffness. Since stiffness and strength are somewhat related, it follows that the structure should have sufficient strength to satisfy the performance. Also, in some cases the performance criteria may be that the structure should remain elastic, which is a specific strength requirement.

Compliance with some performance criteria may be easy to calculate, perhaps requiring additional modelling and analysis, others may require equivalence to prescription based codes such as CAN

CAN/CSA S6-06 for detailing of members, and in some cases compliance may require reference to research material or testing.

This supplement defines two seismic event levels, three bridge classes and five bridge types ranging from simple to special. For the different bridge classes and types, and different seismic levels, a description of the expected performance criteria is given, followed by the required minimum analysis methods to be used.

2. SEISMIC EVENT LEVELS

In this proposal only two levels of earthquake are considered, a design level which may entail damage for some bridge classes, and a service level. The design level earthquake is based on a hazard probability of exceedance of 2% in 50 years, while the service level earthquake is based on the probability of exceedance of 10% in 50 years.

2.1. Design Earthquake

The design level earthquake is defined as the 5% damped uniform hazard seismic response spectrum, $S_a(T)$, for a probability of exceedance of 2% in 50 years (2475 year return period) as defined in the current National Building Code of Canada (NBCC2010) for firm ground conditions. $S_a(T)$ is modified by foundation factors that account for local site soil conditions to give S(T), the design spectral acceleration.

2.2. Service Earthquake

The service level earthquake is defined as the 5% damped uniform hazard seismic response spectrum, S(T), for a probability of exceedance of 10% in 50 years (475 year return period).

3. BRIDGE CLASSIFICATION

Depending on their location and major usage, bridges are classified into three different performance categories that will be specified by the MoT. The expected performance of each category is:

- Lifeline (LL) bridges – expected to remain serviceable for normal traffic after the design level earthquake
- Major Route (MR) bridges – should be able to carry emergency equipment shortly after the design level earthquake, and can be restored to full capacity in a short time to facilitate economic recovery
- Other (OR) bridges – must not collapse under the design level earthquake but may be unserviceable and require major repair or demolition

3.1. Bridge Classification Performance Levels

Table 1 presents, for each bridge classification, the expected performance target for both the design and service level seismic event.

Table 3.1. Bridge Classification Performance Levels

Earthquake level	Performance level		
	OR	MR	LL
Design	Life Safety	Extensive	Repairable
Service	Extensive	Repairable	Minimal

3.1.1. Performance level description

Table 3.2 describes the damage and service limits associated with the different performance levels.

Table 3.2. Performance Level Description

Performance level	Description of damage and service limits
Minimal	Bridge is fully serviceable for normal traffic. Stresses cannot exceed yield, and concrete strains should be limited to 0.003. Spalling of concrete should not occur and residual cracks in concrete members should not exceed the normal crack widths allowed for temperature and creep effects. Residual displacement of the superstructure is not allowed.
Repairable	Bridge can be used for emergency traffic, and repairs can be made without closing the entire bridge allowing normal service within a short time. Inelastic behavior is permitted. Spalling of concrete and buckling of primary members is not permitted. Bracing members can buckle as long as stability is maintained. Members should not need to be replaced and should be able to be repaired in place. Restrainers must not yield. Foundation movements that result in slight misalignment of the spans or settlement of some piers that do not interfere with normal traffic flows, are allowed if repairs can bring the structure back to the original alignment. Drift ratios should be less than 0.5%, and should not impair the normal operation of the bridge.
Extensive	Inelastic behavior is expected and the bridge can be used for restricted emergency traffic after inspection, and with repairs can be restored to full service. Members can have extensive visible damage, such as spalling of concrete and buckling of braces. Buckling of reinforcement is not permitted. Decks may have loss of bearings but should have adequate remaining seat length to carry emergency traffic. Ground lateral and vertical movements must not exceed those that would prevent the bridge superstructure from being brought back to the original alignment.
Life Safety	Bridge spans remain in place but the bridge may be unusable and may have to be extensively repaired or replaced. The structure must be designed to not collapse and persons on the bridge should be able to exit safely. No limit on stresses or strains provided the members can continue to support the bridge dead plus 30% live loads, including P-delta effects, without collapse. Ground lateral and vertical movements are not restricted but must not lead to collapse of the bridge superstructure

4. BRIDGE TYPES

In a mature PBD situation the designer would select the type of analysis needed to ensure the performance of the structure. The MoT feels that for this first PBD specification, minimum levels of analysis for different bridge types and performance levels should be specified.

Bridge type descriptions representing the complexity of the structure which influence the level of analysis that may be needed, are given in Table 4.1.

Table 4.1. Bridge Types

Bridge type	Description
Simple	Straight bridge with one continuous deck from abutment to abutment, including integral abutment bridges. May be continuous over several bents provided the bents are of similar stiffness.
Multi-span	Multi-span bridge of one type with expansion joints. If curved the change in direction should not exceed 20 degrees, or the abutments and piers should not be skewed by more than 20 degrees.
Curved	Multi-span curved bridge of one type with expansion joints, with significant change in direction over the length of the bridge, or with skewed abutments and piers in excess of 20 degrees. Length should be not greater than 100m
Complex	Large bridge that may contain two or more different types of structure, may be curved and/or

	have skewed piers and abutments.
Special	Bridges with special features like base isolation, dampers, unusual foundation treatment, suspension or cable-stays, advanced materials, etc.

5. ANALYSIS

All bridges in seismic hazard regions with S(0.2)≤0.12g need not be analyzed for seismic effects except for seat length requirements and lateral restrainer capacity specified in the current Canadian Highway Bridge Code (CAN/CSA-S6-06). The design should incorporate capacity design principles as much as possible so as to avoid any brittle failures from the seismic forces.

In seismic hazard regions with 0.12g≤S(0.2)<0.35g, simple and multi-span major (MR) route bridges, and all other (OR) bridges in seismic hazard regions with S(0.2)>0.12g, may be designed using CAN/CSA-S6-06 with modifications specified in the MoT Bridge Standards and Procedures Manual. The other bridges must be designed using the performance based design philosophy described herein.

5.1. Minimum Level of Analysis

For different bridge types and required performance level, a minimum level of analysis is specified. Designers may find it advantageous to use a more refined analysis.

2D analysis means a planar analysis involving vertical and horizontal motions. In most cases it considers only horizontal ground movement. A 3D linear dynamic analysis may be a spectral analysis considering only horizontal spectra, or may involve a time step analysis with ground motions applied at the supports. A 3D nonlinear analysis requires a time history analysis with ground motions.

Elastic static **–** application of horizontal inertial forces based on the spectral acceleration at the estimated first mode period, used to determine displacements and forces on bents or abutments of simple bridges.

2D spectral **–** a linear dynamic spectral analysis to determine displacements and member forces considering only horizontal ground motion. Separate analyses may be used in the longitudinal and normal directions of the bridge 4enterline.

3D dynamic – a linear dynamic analysis with horizontal ground motions applied in both horizontal directions. Emphasis is on determining the relative displacements of the adjacent sections of the bridge, and should consider the effects of differing ground motions along the length of the structure.

3D nonlinear dynamic – this analysis would normally be used to check that the performance requirements have been met and would include nonlinear member properties for all elements expected to undergo inelastic response. Vertical ground motions may need to be considered.

PO – pushover **–** a nonlinear static analysis that displaces the structure to the maximum calculated displacement from previous analyses. Nonlinear member rotations and strains are checked against the performance requirements.

Ground motion spectra and time history records to be used in the above analyses are discussed in Section 7.

The minimum level of analysis for the different types of bridges is given below in Tables 5.1-5.3.

Table 5.1. Minimum Level of Analysis for Other (OR) Bridges

Other(OR) bridges	Bridge type

Earthquake level	Performance level	Simple	Multi-span	Curved	Complex	Special
Design	Life Safety	Elastic static, PO	2D spectral, PO	3D dynamic, PO	3D nonlinear dynamic	MoT approved
Service	Extensive	Elastic static	2D spectral	3D dynamic	3D nonlinear dynamic	MoT approved

Table 5.2. Minimum Level of Analysis for Major Route (MR) Bridges

Major Route (MR) bridges		Bridge type				
Earthquake Level	Performance level	Simple	Multi-span	Curved	Complex	Special
Design	Extensive	Elastic static, PO	2D spectral, PO	3D dynamic, PO	3D nonlinear dynamic	MoT approved
Service	Repairable	Elastic static	2D spectral	3D dynamic	3D nonlinear dynamic	MoT approved

Table 5.3. Minimum Level of Analysis for Lifeline (LL) Bridges

Lifeline (LL) bridges		Bridge type				
Earthquake Level	Performance level	Simple	Multi-span	Curved	Complex	Special
Design	Repairable	Elastic dynamic PO	2D spectral, PO	3D dynamic, PO	3D nonlinear dynamic	MoT approved
Service	Minimal	Elastic dynamic	2D spectral	3D dynamic	3D nonlinear dynamic	MoT approved

6. EARTHQUAKE LOAD COMBINATIONS

For the elastic static and the 2D spectral types of analyses, which are carried out separately in each of the two orthogonal principal directions, the forces or displacements from one direction should be combined with 30% of the results from the second direction. This should be repeated with 100% of the forces or displacements in the second direction combined with 30% in the other. Use the maximum of the two responses.

When 3D spectral analyses is used, the input should include the full design spectrum applied separately in both directions, or in all directions if vertical ground motion is to be included, and then use the CQC (or RSS if appropriate) method to combine the results from each direction.

3D dynamic analyses, including nonlinear dynamic analyses, that use horizontal ground motion pairs whose geomean spectra has been scaled to fit the design spectrum, need no further combining. When vertical ground motion is required to be considered, use the same scaling factor for the vertical component of the record set that was used on the horizontal records. When ground motion records that have individually been scaled or modified to fit the design spectrum are used, separate analyses should be run, one with the modified record in one direction and 30% of the modified record in the orthogonal direction. A second run should be performed with the full modified record in the second direction and 30% of the record in the orthogonal direction. When vertical ground motion is to be considered, 30% of the modified record should also be used in the vertical direction, plus a third analysis with 100% in the vertical direction and 30% in the two horizontal directions. Use the maximum of the responses.

6.1. Gravity and Earthquake Load Combinations

Live loads are not included in the horizontal inertia mass as vehicles generally do not respond in the same manner as the bridge. However a portion of the live load should be included in the gravity loads.

During the earthquake the load combination should be: 1.0D + 1.0EQ + 0.5L. After the earthquake, bridges with Life safety level damage are expected to not have collapsed but there is no further requirement for carrying loads. For the Extensive damage level the bridge should be able to carry loads of 1.25D plus 0.5L, for the Repairable damage level the bridge should be able to carry 1.25D plus 0.75L, and for the Minimal damage level 1.25D plus 1.0L.

7. HAZARD SPECTRUM

In other than a few regions the earthquake hazard in Canada is given in terms of a uniform hazard spectrum (UHS). De-aggregation plots are available that give an indication of the earthquake magnitudes and distances that contribute most to the hazard. The hazard is usually given for a site deemed ‘firm ground’, and must be adjusted to account for the local foundation conditions.

The UHS is comprised of motions from both near and far field earthquakes and as such is not representative of the spectra that would be given by a single earthquake over the full range of periods. Earthquakes that produce a spectrum that matches the UHS at a particular period would be expected to have spectra smaller than the UHS for other periods. Such a reduced spectrum has been defined as the conditional mean spectrum (CMS). The UHS will always be larger than the CMS, and is therefore a conservative spectrum to use, however the CMS depends on the period where it is to match the UHS so several different CMS may need to be considered if the structure has many important periods.

Probabilistic uniform hazard vertical ground motion spectra are generally not available. Unless a site specific analysis is made for the vertical ground motion spectrum, it should be taken as 2/3 of the horizontal design spectrum.

8. GROUND MOTION RECORDS

Ground motion records should be representative of the magnitude and distance of earthquakes that contribute most to the design spectra. De-aggregation plots give the contribution of different magnitude earthquakes, at different distances from the site, to the UHS, and can be generated for spectral values at different periods. In many cases a mean magnitude and mean distance can be identified as contributing most to the site hazard. If de-aggregation of the hazard at the site shows a bipolar distribution, that is, if a second magnitude/distance scenario contributes appreciably to the hazard, then two different magnitude/distance combinations should be considered.

Any time history analysis should consider at least 3 ground motion records with the most adverse result used. If the analysis uses 7 or more records, then an average value of the results can be used. The spectra of the records should provide a reasonable fit to the design spectra over a period range of interest of at least $0.2T_1$ to $1.5T_1$, where T_1 is the fundamental period of the bridge. If important portions of the bridge have periods lower than $0.2T_1$, then the range of the spectral match should include these periods.

For a 3D analysis where motion in both horizontal directions is to be applied at the same time, pairs of records that maintain the correlation between the two directions should be used. The geometric mean spectrum of the pair of horizontal records should be used in selecting records that best match the design spectrum. The geometric mean spectrum, is defined by Eqn. 8.1 where S_x and S_y are the spectra of the two orthogonal records.

$$S_g=\sqrt{(S_x S_y)} \tag{8.1}$$

For a 2D analysis where motion in only one direction is to be applied at any time, the average of the individual records only need match the UHS or CMS.

Spectral matching requires that over the selected period range, the scale factor to be applied to each record set should be calculated so that the scaled spectra minimizes the square of the difference from the design spectrum over the period range of interest.

For 2D analyses, records that have been modified to match the design spectrum may be used in lieu of scaled records. Modified records do not maintain the original correlation between pairs of horizontal records, and so should only be used for 3D analyses if it is not possible to get enough suitable record pairs. In such a case both horizontal records must be modified to match the design spectrum.
When using scaled earthquake pairs the spectrum in one direction will usually be smaller than the other in the period range of interest. When assembling a set of earthquake records care should be taken to have roughly half the records with, say the smaller spectrum, in the same analysis direction.

8.1. Vertical Earthquake Records

For analyses that use records scaled so that the geomean spectrum matches the design spectrum, the same scale factor should be used for the vertical records. However care should be taken that the magnitude and distance of the records represent the main seismic hazard and the local soil conditions, especially if soft soil conditions are present at the site. For analyses that use records modified to fit the design spectrum the usual practice has been to use 2/3 of the horizontal record as the vertical record.

9. RESTRAINERS AND SEAT LENGTHS

The purpose of restrainers is to maintain the integrity of the bridge, in particular, to prevent spans from falling from the supports. Restrainers are generally of two types; rigid or very stiff restrainers in the lateral direction to the bridge, and flexible in the longitudinal direction. Longitudinal restrainers are slack to allow unrestrained thermal or creep movements, but act in tension to restrict larger opening movement between adjacent bridge sections or between the bridge and the abutments. The required strength and stiffness of the restrainers, and deformation capacity, are very dependent on the seat length.

In the absence of a rigorous analysis the seat length requirements specified in CAN (the same as AASHTO) should be followed. The strength of the restrainers should be as specified in CAN, with the additional proviso that the stiffness of the restrainers should be such that the strength of the restrainers is reached before the movement of the span exceeds the seat length.

If the restrainers are incorporated into a 3D dynamic analysis, or in the case of longitudinal restrainers in a straight bridge a longitudinal 2D dynamic analysis, the seat length should be 1.5 times the maximum calculated longitudinal displacement relative to the pier/abutment or the opening of an expansion joint, but should not be less than 200mm.

Lateral restrainers are generally very rigid and designed to restrain all movement. They must be able to resist a maximum lateral force, increased by 50% from the analysis or the maximum lateral capacity of the bridge, and must not fail in a brittle manner. In low seismic regions where the lateral capacity of the bridge may be much greater than the lateral force from the analysis, the lateral design force should be a minimum force of 1.5 times the design peak spectral acceleration times the supported mass.

10. FOUNDATION TYPES AND MINIMUM ANALYSIS METHODS

Different site conditions and different foundation types may require different minimum analysis procedures. Listed below are four types of foundations which are likely to be used on different ground conditions, along with possible minimum analysis methods. There may be other combinations of foundation type and ground conditions where the minimum analysis method needs to be addressed on a job specific basis.

10.1. Abutments or Piers on Spread Footings

These foundations would normally only occur at sites with firm ground. No special analysis of the foundations would be needed except that ‘soil’ springs may be incorporated into the bridge analysis model, especially in the longitudinal direction at the abutments.

10.2. Abutments or Piers on Pile Supports

If the footings are on poor soil and the piles are used to provide both vertical and lateral support, then the bridge analysis should at a minimum include ‘soil’ springs (or finite elements) along the piles and/or footings. Large bridges supported on deep piles may require a full soil/structure interaction analysis.

10.3. Pile Bents

Pile bents are generally used on soft soil or river sites and consist of piles extending out of the ground to a pier cap that holds the beams supporting the bridge deck, and form the lateral resisting system for the bents. As such they are part of the bridge system as well as the foundation. As a minimum the analysis should include ‘soil’ springs and be consistent with the bridge minimum analysis, and at least 2D dynamic, but a more extensive soil/structure interaction analysis may be required on a job specific basis.

10.4. Caissons

Caissons generally only occur with large bridges on soft soil or water sites and have a large influence on the seismic motion of the bridge superstructure. As a minimum the analysis should include ‘soil’ springs and be consistent with the bridge minimum analysis, but a more extensive soil/structure interaction analysis may be required on a job specific basis.

10.5. Liquefaction

For sites with the potential of liquefaction, estimates must be made of the expected post-earthquake displacements of the footings and abutments, as well as the foundation resistance available during and after the event. The resistance factor used should consider the performance expectation and the technology that was used to determine the soil resistance. Differential movements between footings and/or abutments must be consistent with the prescribed allowable performance level for the structure.

A simplified analysis may be justified, and if used must consider reductions in soil strength and use input time histories in a Newmark type analysis.

The bridge should be analysed and designed for the ground motions under the assumption that liquefaction will not occur. If liquefaction does take place the ground accelerations at the level of the foundations are generally reduced but the periods lengthened, which may result in larger displacements in the structure.

The assessment of liquefaction is usually based on empirical relations considering horizontal motions only. However, vertical ground motions may have an influence on the lateral displacements of fills and embankments.

10.6. Slopes and Embankments

Movements of the embankments either through settlement or slope failure must not render the bridge unusable for the prescribed performance. The level of analysis required to assess the embankment movements should be consistent, but not necessarily the same as the minimal level of analysis prescribed for the bridge.

Seismic loading-induced deformation analysis may be required, for abutment sections, any pier locations at or near sloping ground, and approach embankments. The deformation analysis may consider input ground motion time-histories and should take into consideration the anticipated reductions in shear strength and stiffness of the soil due to strong shaking. These analyses shall be performed using a computer code that is capable of taking into consideration non-linear soil behaviour, pre- and post-liquefaction stress-strain-strength behaviour of soils, soil/structure interaction effects, and time domain base input excitations.

A simplified analysis may be justified, and if used must consider reductions in soil strength and use input time histories in a Newmark type analysis.

Vertical ground motions may increase movement and should be considered.

10.7. Retaining Walls

Retaining walls, where failure from seismic ground motion would impact the bridge performance or the use of the bridge, must be considered and be consistent with the performance criteria for the bridge.

Dynamic soil/structure interaction analysis shall be performed for retaining walls supporting 5 m or more of soil or those walls supporting abutment foundations. Analysis software to be used shall be capable of taking into consideration non-linear soil and structure behaviour and time history input ground motions to demonstrate that the seismic performance criteria are satisfied. The de-aggregation of soil stiffness should be considered. Vertical ground motions may increase movement and should be considered.

10.8. Ground Movements from Adjoining Sites

Estimates of ground movements from adjoining sites that would impact the bridge performance should be estimated and brought to the attention of the Ministry.

REFERENCES

Canadian Highway Bridge Design Code CAN/CSA-S6-06 (CHBDC)
BC Ministry of Transportation Supplement to the Canadian Highway Bridge Design Code, CAN/CSA-S6-06 (Supplement to CHBDC S6-06)