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4.1 Scope

Add the following:

Commentary: *Chapter 4 Seismic Design of S6-14 has made a major shift in philosophy toward performance-based design, which is in keeping with current BC practice for bridges.*

4.2 Definitions

Commentary:

Capacity design *is a seismic design method in which the Designer selects, designs and details a primary lateral load resisting system to behave in a ductile and predictable manner while supporting specified gravity loads at deformations well beyond the elastic limits of the lateral load resisting system.*

Traditionally the capacity design approach involved an explicit selection of a plastic mechanism as the lateral load resisting system with pre-selected plastic hinges (structural fuses) to allow the designer to control and limit forces in the non-yielding regions or components of the ductile substructure. Controlling the capacities of structural fuses allows the design forces on both the fuses and on adjacent structural components to be controlled. Detailing and proportioning the fuses and the adjacent components delays brittle failure modes until large post-elastic deformations occur, providing a significant degree of structural integrity and resilience to the bridge system for seismic loads beyond the minima specified by the code. The method may also be applied to base-isolated bridges (where isolation bearings become the structural fuses) or to other energy-dissipating lateral load resisting systems. Elastic forces calculated from static or dynamic analyses may be acceptable in the design of the lateral load-resisting system within S6-14 and this Supplement, but such forces do not constitute 'capacity protection' within a capacity design approach. See specific requirements under Clause 4.4.10.4.

Capacity-protected element *- the critical structural component that is being protected from damage by using the limited and controlled structural capacity of ductile elements within the lateral load-resisting system.*

Probable resistance**:** *The combined effects of probable resistances (overstrength factor >1.0, see Clauses 4.4.10.4.2 and 4.4.10.4.3) with expected material properties (see Clause 4.7.2) can be considered as equivalent to the over-strength capacity of structural components as described in previous codes. The term "over-strength" is not used in S6-14 but is conceptually important in a capacity design approach.*

***Static Pushover analysis** - an inelastic static analysis involving a step-by-step force-deformation procedure in order to identify the local and global inelastic behaviour and failure modes of the lateral load resisting system*

Pushover analyses are used to determine both capacity design demands and to assess structural behaviour and damage at each stage of inelastic deformation of the lateral load resisting system. Section capacities can account for degradation with increasing ductility demands, and the local deformations and strains allow for damage and performance assessments at all specified earthquake levels.

Add the following definitions:

Extended pile bent – Gravity and lateral load resisting substructure comprising piles that extend above grade without an at-grade pile cap, connecting directly to the pier cap beam supporting the bridge superstructure. Where “pile bent” is used in this chapter it may be interpreted as an extended pile bent.

Seismic performance category (SPC): A category assigned to a bridge that affects the requirements for design approach (FBD or PBD), analysis (See Clause 4.4.4 and Table 4.10) and detailing.

Sign structures – Structures supporting signs for road direction, tolling equipment or messages that span or cantilever over a roadway.

4.3 Abbreviation and Symbols

4.3.2 Symbols

Commentary:

P_f within a capacity design approach can account for plastic behaviour in the lateral resisting system.

4.4 Earthquake effects

4.4.2 Importance categories

Replace the first sentence with:

The Ministry will designate bridges into one of the following three importance categories:

***Commentary:** Low Volume Road (LVR) bridges are typically designated as "other" bridges unless otherwise specified by the Ministry.*

4.4.3 Seismic hazard

4.4.3.1 General

Delete the last sentence of the first paragraph and replace with the following:

Spectral values shall be adjusted to reflect local site conditions in accordance with Clause 4.4.3 to give the design spectral values. Design spectral values may also be obtained using site response analysis with consent of the Ministry. The spectra from site response analysis shall not be less than 80% of the code based spectra.

Delete the 4^{th} paragraph.

4.4.3.2 Site properties

Commentary: *Update No. 1 to S6-14 was published in April 2016 and shall apply.*

4.4.3.3 Site coefficients

Commentary: *Update No. 1 to S6-14 was published in April 2016 and shall apply.*

4.4.3.6 Time-history input motions

Time history input motions used in the design are subject to the consent of the Ministry.

4.4.4 Seismic Performance Category

Change Table 4.10 SPC from 2 to 1 for Row 1 for Lifeline Bridges (See modified Table 4.10 following).

Table 4.10
Seismic performance category based on 2475 year return period spectral values
(See clause 4.10.3)

For T < 0.5 s	For T > 0.5 s	Seismic Performance Category	
		Lifeline bridges	Major-route and other bridges
S(0.2) < 0.20	S(1.0) < 0.10	1	1
0.20 <= S(0.2) < 0.35	0.10 <= S(1.0) < 0.30	3	2
S(0.2) >= 0.35	S(1.0) >= 0.30	3	3

Add note to Table 4.10 as follows:

For lifeline bridges in SPC 1, detailing of structural elements shall adopt requirements for SPC 2 as a minimum.

Commentary: *As published by CSA, and considering also Table 4.11, all Lifeline bridges in BC (and Canada) regardless of seismic hazard would require explicit demonstration of seismic performance through PBD. Values for S(0.2) < 0.2 and for S(1.0) < 0.1 are considered unduly low for many bridges to benefit from the analyses and methods of PBD methods. At low levels of seismic hazard, a bridge's seismic performance would have little or no post-elastic behaviour, such that the bridge design focus should not be on plastic design methods. The Ministry may require PBD on specific projects.*

4.4.5 Analysis and design approach

4.4.5.1 General

Delete the reference to "Clause 4.4.3.5" in the last sentence and replace with "Clause 4.4.10.2".

Add the following sentence:

Sign structures in seismic performance categories 2 and 3 require that seismic performance be demonstrated for a no-collapse requirement at a 2% in 50-year hazard (2,475 year return period). *See also Clause 4.4.6.1.*

Commentary: *Collapse prevention for sign bridges should be demonstrated using displacement-based approaches and considering local plastic behaviour and buckling to demonstrate performance. Applicable clauses within and cross-referenced from Clause 4.8.4 should be applied.*

4.4.5.2 Single-span bridges

4.4.5.2.1 Analysis requirements

Replace the first sentence with

In Seismic Performance Categories 2 and 3, all bridges except single span bridges having a skew angle less than 20^o and a maximum subtended angle of 30^o shall be analyzed and designed to address the seismic behaviour resulting from the geometric irregularities.

4.4.5.3 Multi-span bridges

4.4.5.3.1 Analysis requirements and design approach

For Table 4.11: Change the last sentence in the title of the Table to:

(See Clause 4.4.7 for FBD requirements).

Replace Table 4.12 with the following:

Seismic Performance Category	Lifeline Bridges		Major Route Bridges		Other Bridges	
	Irregular	Regular	Irregular	Regular	Irregular	Regular
1	No seismic analysis required					
2	EDA, ISPA and NTHA	EDA and ISPA	EDA and ISPA	ESA	EDA	ESA
3	EDA and ISPA NTHA	EDA and ISPA	EDA and ISPA	EDA	EDA	ESA

Add the following:

As a minimum, the following geotechnical engineering input shall be incorporated in the structural analysis methods described in Table 4.12:

Elastic Static Analysis (ESA): These analyses may be carried out on structural model(s) without rigorous treatment of soil-structure interaction but should include effects of foundation flexibility important to the global structural response. The seismic demand may be based on free-field ground surface or near-surface (as appropriate to the foundation system) response spectrum established using either code factors or wave propagation (1D or 2D) analysis as consented to by the Ministry. Where the benefits of site-specific site response analyses are sought, site characterization consistent with “a high degree of site understanding” shall be undertaken, and ground motions that represent the site and hazard shall be determined.

For Class F sites, the inertial loads shall be established based on the Geological Survey of Canada (GSC) response spectra adjusted for site conditions as per the shear wave average velocity classification, or using spectra from site response analysis.

Elastic Dynamic Analysis (EDA): These analyses shall be carried out on structural model(s) with an appropriate treatment of soil-structure interaction that capture as a minimum the effects of foundation flexibility important to global structural response. The seismic demand shall be based on site-specific free-field response spectrum or time-history records computed at an elevation determined by the structural and geotechnical engineers. The applicable free-field response spectrum shall be established using either code factors or wave propagation (1D or 2D) analysis as consented to by the Ministry utilizing equivalent linear or non-linear method of analysis. Where the benefits of site-specific site response analyses are sought, site characterization consistent with “a high degree of site understanding” shall be undertaken, and ground motions that represent the site and hazard shall be determined.

For Class F sites, the inertial loads shall be established based on the Geological Survey of Canada (GSC) response spectra adjusted for site conditions as per the shear wave average velocity classification, or using spectra from site response analysis.

Inelastic Static Pushover Analysis (ISPA): These analyses shall be carried out on a full or partial model of the bridge system incorporating the effects of foundation flexibility using methods outlined in Clause 4.6.4.

Where applicable(e.g. liquefaction-induced lateral spreading or settlements), the effects of kinematic loading from inelastic ground deformations on the structure shall be evaluated and combined with the displacement and other effects of inertial loading using the combinations described below:

- 100% kinematic demands
- 100% inertial demands
- 50% inertial demands + 100% kinematic demands

In cases where soil softening does not reduce the inertial effect, then a special assessment shall be undertaken to develop an appropriate combination of inertial plus the applicable kinematic effects.

For Class F sites, the inertial loads shall be established based on the GSC response spectra adjusted for site conditions as per the Vs classification, or using spectra from site response analysis.

Non-linear Time-history Analysis (NTHA): These analyses shall be carried out on a full or partial model of the bridge system incorporating the non-linear behaviour of foundation soils and foundation elements. Computer software used for this purpose shall have the capability to incorporate non-linear soil effects, pre- and post-earthquake stress-strain-strength characteristics of soils, and non-linear structure effects. These analyses shall be either 2D or 3D. Unless otherwise specified by the Ministry, analyses shall be carried out for all input ground motions defined in Clause 4.4.3.6.

***Commentary**: Tables 4.12 and 4.13 apply to structural analyses including appropriate modelling for important soil-structure interaction effects in all analysis types. They do not refer to site response analyses used for seismic hazard considering soil behaviours.*

Foundation flexibility can be important in ISPA, whether for stand-alone piers or for piers within bridge systems as it can affect the location and progression of plastic hinging, on local ductility demands at hinges, and on demand calculations for capacity protected elements.

Kinematic demands include the effects of liquefaction-induced ground deformations, for example lateral spreading or support settlements. The combinations provided are intended for sites where kinematic demands are induced by liquefaction which reduces the soils ability to transmit ground motions to the structure. Where this is not the case, then a special assessment is required to develop an appropriate combination of inertial plus the applicable kinematic effects. In lieu of an explicit effective stress and non-linear coupled approach to these combined effects, some allowance for concurrent effects is appropriate.

Multiple support inputs are difficult to predict in British Columbia owing to the limited information on known faults. These effects may also provide a net reduction in structural response. Project specific seismic specifications will be provided for important or major bridges when needed.

4.4.6 Performance-based Design

4.4.6.1 General

Replace third paragraph with:

Lifeline bridges in SPC 2 and 3 shall require independent peer review unless stated otherwise in project specifications.

4.4.6.2 Performance levels

Replace Table 4.15 with the following:

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*	Immediate	Minimal	Service Limited*	Repairable*
5% (975 years)	Immediate	Minimal	Service Limited*	Repairable*	Service Disruption*	Extensive*
2% (2475 years)	Service Limited	Repairable	Service Disruption	Extensive	Life Safety	Probable replacement

* Optional performance levels unless required by the Ministry.

Commentary: *S6-14 mandates a higher seismic hazard (2475 year levels) than S6-06. This change is consistent with hazard levels in NBCC 2015. S6-14 also introduced PBD. Damage levels as tied to service expectations are believed to have been unduly conservative for modern, well-detailed columns in ductile substructures. This was true in particular for the "none" and "minimal" damage descriptions. Accordingly, adjustments to performance requirements in Table 4.15 (above) and for damage descriptors in Table 4.16 (below) are adopted.*

4.4.6.3 Performance criteria

Table 4.16:

Replace description for "Minimal Damage" to:

Minimal Damage

- General: Bridge shall sustain minor damage that does not affect the performance level of the structure.
- Concrete Structures: Concrete compressive strains shall not exceed 0.006 and flexural reinforcing steel strains shall not exceed 0.010.
- Steel Structures: Steel strains shall not exceed yield (see Clause 10.5.3.3). Local or global buckling shall not occur.
- Connections: Connections shall not be compromised.

- Displacements: Residual displacement, settlement, translation or rotation, of the structure or foundations, including retaining and wing walls, shall not compromise the performance level.
- Bearings and Joints: Shall not require replacement except for possible damage to joint seal.
- Restrainers: Negligible damage and no loss of displacement capacity to restraining systems or connected elements.
- Foundations: 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.

Replace description for “Repairable Damage” to:

Repairable Damage

- General: The bridge may experience inelastic behaviour, however primary members shall be repairable in place and shall be capable of supporting the dead load plus live load corresponding to the service performance criteria during repairs.
- Concrete Structures: Tensile rebar strains shall not exceed 0.025.
- Steel Structures: Buckling of primary members shall not occur. Secondary members may buckle provided that stability is maintained. Net area rupture of primary members at connections shall not occur
- Connections: Primary connections shall not be compromised.
- Residual displacements including settlement, translation or rotation of the structure or supports, including abutments, retaining and wing walls shall not compromise the service and repair requirements of the bridge.
- Bearings and Joints: Replacement of elastomeric bearings is permitted provided that service requirements are not compromised. Damage to other structural bearings shall not compromise the integrity of the structure nor compromise the service requirements. Replacement of joints is permitted.
- Restrainers: Restrainers shall not rupture and shall retain their ability to prevent span loss in aftershocks. Damage to restrainer supporting elements such as end diaphragms or substructure shall not require bridge closure to repair.
- Ground deformations shall be mitigated such that permanent foundation offsets are small and repair objectives specified above can be met. Foundation offsets shall be limited such that repairs can bring the structure back to the original operational capacity.

Commentary:

The requirements for demonstration of aftershock capacity have been deleted at this time since there are no generally accepted methodologies for this type of assessment.

In general, superstructures, ductile substructures, restrainers and foundations designed to S6-14 PBD methods are considered to have inherently met expectations for aftershocks without additional assessment. This is because the design methods and detailing result in a *robust structure which retains essentially its full capacity after the design event and is capable of sustaining multiple additional cycles of seismic loading.*

ATC-49 provides guidance in determining performance limits for pile foundations.

Replace description for “Extensive Damage” to:

Extensive Damage

- General: Inelastic behaviour is expected. Members may have extensive visible damage, such as spalling of concrete and buckling of braces but significant strength degradation is not permitted. Members shall be capable of supporting the dead load plus 1 lane of live load in each direction (to account for emergency vehicles), including P-delta effects, without collapse.
- Concrete Structures: Extensive concrete spalling is permitted but the confined core concrete shall not exceed 80% of its ultimate confined strain limit. Reinforcing steel tensile strains shall not exceed 0.05.
- Steel Structures: Global buckling of gravity load supporting elements shall not occur.
- Connections: There may be significant joint distortions but damaged connections must maintain structural integrity under gravity loads.
- Structural displacements: There may be permanent structural offsets as long as they do not prevent use by restricted emergency traffic after inspection or the bridge, nor preclude return of full service to the bridge after major repairs.
- Bearings and Joints: The bearings may be damaged or girders may become unseated from bearings, but girders shall have adequate remaining seat length and connectivity to carry emergency traffic. Bearings and joints may require replacement.
- Restrainers: Restraining systems might suffer damage but shall not fail.
- Foundations: Foundation lateral and vertical movements must be limited such that the bridge can be used by restricted emergency traffic. Foundation offsets shall be limited such that repairs can bring the structure back to the original operational capacity.

Commentary: *The requirements for demonstration of aftershock capacity have been deleted at this time since there are no generally accepted methodologies for this type of assessment.*

In general, superstructures, ductile substructures, restrainers and foundations designed to S6-14 PBD methods are considered to have inherently met expectations for aftershocks without additional assessment. This is because the design methods and detailing result in a robust structure which retains essentially its full capacity after the design event and is capable of sustaining multiple additional cycles of seismic loading.

ATC-49 provides guidance in determining performance limits for pile foundations.

Replace description for "Probable Replacement" to:

Probable Replacement:

- General: Bridge spans shall remain in place but the bridge may be unusable and may have to be extensively repaired or replaced.
- Concrete Structures: Damage does not cause crushing of the confined concrete core. Reinforcing steel tensile strains shall not exceed 0.075, except that for steel reinforcing of 35M or larger the strains shall not exceed 0.060.
- Extensive distortion of beams and column panels may occur.
- Members shall be capable of supporting the dead plus 30% live loads, excluding impact, but including P-delta effects, without collapse
- Fractures at some moment connections may occur that don't significantly increase the risk of collapse. Shear connections shall remain intact.
- Displacements: Permanent offsets shall be limited such that the bridge can be evacuated safely.
- Foundations: Foundation movements shall not lead to collapse of the bridge superstructure nor prevent evacuation.

4.4.6.4 Performance Criteria for Walls, Slopes and Embankments

The following seismic performance criteria shall be met for the design of retaining walls, slopes and embankments:

Category	Retaining Walls	Slopes and Embankments
Lifeline	• No collapse of retaining wall during and following 975-year ground motion • 100% of lanes in close proximity to the bridge are available for use following 975-year ground motion • 50% of lanes away from the bridge are available for use following 975-year ground motion • Permanent wall lateral deformations shall be such that the service level performance and damage level performance requirements for Structures are met.	• 100% of lanes in close proximity to the bridge are available for use following 975-year ground motion • 50% of lanes away from the bridge are available for use following 975-year ground motion

Major-route	• No collapse of retaining wall during and following 975-year ground motion • Permanent wall lateral deformations shall be such that the service level performance and damage level performance requirements for Structures are met.	• Factor of Safety against slope failure under static loading as per Table 6.2b. • Pseudo-static factor of safety against slope failure = 1.1 under 975-year ground motion
Other	• No collapse of retaining wall during and following 475-year ground motion • Permanent wall lateral deformations shall be such that the service level performance and damage level performance requirements for Structures are met.	• Factor of Safety against slope failure under static loading as per Table 6.2b. • Pseudo-static factor of safety against slope failure = 1.1 under 475-year ground motion

Note: As a minimum, the distance defined as “close proximity to the bridge” shall be taken as the length of an approach embankment equal to a horizontal distance that is twice the height of the embankment or retaining wall, as the case may be. This distance may be altered by the Ministry in project-specific requirements.

4.4.7 Force-based Design

4.4.7.1 General

Add the following paragraph:

For regular bridges of slab, beam-girder, or box girder construction, a detailed analysis of earthquake effects on superstructure components is not required. However, lateral analysis and related design of cross-frames or diaphragms between girders at the abutments and piers, and of bearings, bracing connections and connections between the superstructure and substructure are required.

4.4.7.2 Response modification factor

Delete the last paragraph.

4.4.9 Load factors and load combinations

4.4.9.2 Earthquake Load Cases

Commentary:

Orthogonal load combinations in this section were developed primarily for force-based design approaches on piers, but should also be used to make allowances for coupling of displacement demands and response in orthogonal directions. Displacement and force demands are commonly calculated and assessed in each direction separately.

These directional combinations were not calibrated for abutment or retaining wall design. Abutments and walls are normally designed using earthquake loads in each direction separately. For skewed abutments it is common to check abutment stability using pressures perpendicular to the ballast wall. This approach is acceptable, including for integral abutment bridges, for skew angles 20^o or less. For higher skew angles, concurrent directional combinations in orthogonal directions should be investigated more explicitly. Structurally, the effects of displacements normal to the abutment should be considered in detailing for seat lengths and global structural response.

4.4.10 Design forces and support lengths

4.4.10.1 General

In second paragraph, delete the sentence: “These restraint forces need not apply if the requirements of clauses 4.4.3.5 are satisfied.

Add the following:

For bridges without transverse seismic shear restraint, the transverse support length from the edge of the girder to the transverse face of pier or abutment shall be in accordance with the “N” dimension from Clause 4.4.10.5.

Commentary*: Clause 4.4.10.1 refers specifically to connection forces between superstructure and substructure, when structural connections are used (e.g. through bearings or separate restrainers). They are prescriptive for that purpose alone. They apply when seat lengths are less than prescribed as a means to prevent loss of span failures. Neither these connection forces nor the seat lengths prescribed in Clause 4.4.10.5 are applicable to integral or semi-integral abutment bridges in which the superstructure – integral with the ballast wall or the entire abutment – is restrained from movements by soil pressures during earthquakes. In lieu of analyses and calculations to demonstrate that shorter seat lengths are sufficient for integral or semi-integral abutments, the seat lengths of Clause 4.4.10.5 shall be used.*

These connection forces are also not intended to be combined with seismic soil pressure forces on abutments, nor with self-inertia forces from massive concrete abutments. This issue has been investigated recently as part of the AASHTO LRFD code, with a disposition consistent with the above *commentary. See also Clauses 4.4.10.7 regarding hold-down forces and Clause 4.6.5 regarding seismic forces on abutments and retaining walls..*

4.4.10.4 Seismic Performance Category 3

4.4.10.4.2 Modified seismic design forces for force based design

Delete the second paragraph and replace with the following:

Seismic design forces for capacity-protected elements shall be designed to 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, as part of an identified plastic mechanism or other predictable mechanism attaining their probable resistance.

Where a seismic lateral load-resisting system relies on elastic forces rather than on capacity design principles to control demands, brittle failure modes in lateral-load resisting elements shall use design seismic forces of 1.25 times the elastic seismic forces (i.e. $R = 1.0$ and $I_E = 1.0$).

Commentary: *Based on S6-14 for the design of capacity-protected elements, the margin of resistance compared to demands of the chosen ductile substructure mechanism, is summarized as:*

$$\phi \mathbf{R}_{nominal} > \phi_{probable} \times \{\mathbf{D}_{expected}\}$$

where

ϕ — *concrete, rebar or steel resistance factors in Chapters 8 and 10*

$\mathbf{R}_{nominal}$ — *section resistance using specified grades for material strengths*

$\phi_{probable}$ — *Phi factor greater than unity as described in Clause 4.4.10.4.3*

$\mathbf{D}_{expected}$ — *Demand calculated using a ductile plastic mechanism (or other predictable and acceptable lateral load-resisting system) using expected material properties as defined in Clause 4.7.2.*

This margin is also to be used for capacity design checks following a performance-based design approach.

Elastic forces may be smaller than those derived from plastic mechanisms but design to such elastic forces is not considered to produce “capacity protection” and may produce a small margin against unexpected brittle failure modes. Components designed elastically require additional conservatism to ensure that brittle failures or collapse would not occur at demand levels marginally greater than the adopted seismic hazard. The 1.25 factor is generally consistent with the approach specified for connection force design in Clause 4.4.10.4.2.

4.4.10.4.3 Yielding mechanisms and design forces in ductile substructures

Delete the third paragraph and its related clauses (i.e. (a) and b)) and replace with the following:

Shear and axial design forces for columns, piers, and pile bents due to earthquake effects shall be as follows:

(a) Shear Force – the shear corresponding to inelastic hinging of the column as determined from static plastic analysis considering the probable flexural resistance of the member and its effective height. 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. If the column foundation is significantly below ground level, consideration shall be given to the possibility of the hinge forming above the foundation due to soil confinement. This is acceptable provided the inelastic hinges are at reasonably accessible and repairable locations.

(b) Axial Force – the axial force corresponding to inelastic hinging of the column in a ductile substructure at its probable resistance.

For cases where elastic design forces are significantly lower than forces derived from capacity design principles, then for capacity-protected elements in accordance with Clause 4.4.10.4.2, shear and axial design forces for ductile substructure elements shall be taken as the unreduced elastic design forces increased by 1.25 times and in accordance with Clause 4.4.9 (i.e R=1.0 and I_E=1.0.)

Commentary: *The Ministry considers "reasonably accessible" to mean less than 2 metres below ground or below mean water or tide level*.

4.4.10.7 Hold-down devices

Replace this clause with:

Bridges in Seismic Performance Categories 2 & 3 shall be vertically restrained unless otherwise consented to by the Ministry. Hold-down devices shall be provided to resist a minimum uplift force of 0.3D or the net uplift force that exists resulting from the tributary dead load (D) multiplied by $\left(F_a S_a\left(0.2\right)-1.0\right)$ whichever is greater. The hold-down devices shall consist of anchored vertical bars and must be of reinforcing steel of Grade 400W, 500W or steel having similar or better rupture strains and ratios of ultimate stress to yield stress.

Where design and detailing explicitly accounts for uplift effects in bridges using seismic isolation systems, supplementary uplift restraint as described in this clause is not required.

Commentary: *Uplift restraint is regarded as a beneficial feature in bridges in zones of high seismic hazard. Alternative hold-down details are subject to Ministry consent. Integral or semi-integral abutment bridges, or bridges with structurally integral superstructure-to-substructure connections, would be* *considered to be held down at the relevant supports if the required capacity were demonstrated.*

4.5.3.5 Static pushover analysis

Add to this clause

The static pushover analysis must be taken to the deformation necessary to identify the full plastic mechanism, expected ultimate displacement capacity, and ultimate failure mode. Displacement demands must capture global bridge response considering the behaviour of the individual pier or support within the global model unless the designer demonstrates that relevant information can be obtained with a local model.

Foundation flexibility must be considered within pushover models to obtain a realistic pattern of hinges and their related deformations.

Commentary: *Static push-over analyses are used to define the sequence of inelastic action in ductile structures, to develop member design forces for 'capacity protection' in ductile substructures, and to assist in defining deformation capacity. They may also be used to assist in defining stiffness and hysteretic properties for use in inelastic dynamic analyses.*

The pushover analysis should be used to identify the expected ultimate failure mode and displacement to identify the margins of reserve and resiliency inherent in the design, and to assist the Ministry in evaluating the design. Local pier models are often adequate for ISPA, but global response effects (e.g. torsion in plan from variations in pier stiffnesses) should also be considered. In some cases, for example integral superstructure-to-pier connections, a push-over model must consider the restraint imposed by the bridge on the local pier response. The model used should be appropriate to capture the important aspects of seismic behaviour.

4.6 Foundations

4.6.2 Analysis methods

Add to first paragraph:

The analysis shall address local site effects, including slope and basin effects where applicable, and effects from or on adjacent infrastructure.

4.6.3 Geotechnical resistance factor

4.6.3.1 Performance-based design

Delete last sentence and replace with:

The consequence factor shall be 1.00.

Commentary: *S6-14 provides resistance factors only for "essentially elastic" performance, for capacity design and for "life safety" performance. It is not the intent that the length or number of piles be increased by forcing the use of* *static resistance factors for intermediate damage states. For these intermediate damage states, performance based design shall apply.*

For preliminary design for axial resistance of deep foundations, one approach could be to modify the static values (Table 6.2) at different seismic performance levels such as follows:

Immediate/Minimal = Static loading value + .1
Service Limited/Repairable = Static loading value + .2
Service Disruption/Extensive = Static loading value + .25
Life Safety/Probable replacement = 1.0

For example, for compression of deep foundations with a Static Pile Test for low/typical/high degree of understanding, resistance factors would be:

Degree of Understanding (See Section 6.5.3.2)	*Resistance Factor from Table 6.2*	*Minimal Damage*	*Repairable Damage*	*Extensive Damage*	*Probable Replacement*
Low	*.5*	*.6*	*.7*	*.75*	*1.0*
Typical	*.6*	*.7*	*.8*	*.85*	*1.0*
High	*.7*	*.8*	*.9*	*.95*	*1.0*

4.6.5 Seismic forces on abutments and retaining walls

Add the following commentary:

Commentary: *This clause considers lateral forces from soils and abutment inertia. It does not require these be combined with forces from superstructure connections specified in Clause 4.4.10.1 and from Clauses 4.4.10.6, 4.4.10.7 and 4.4.10.8. Superstructure connection forces and combinations were recently updated in AASHTO, 2015, and have reverted to <u>not</u> combining superstructure <u>connection</u> forces with substructure seismic soil pressures. Such effects may combine for short durations and may be either detrimental or beneficial to sub-structure components, but are unlikely to increase abutment displacements. For pile-supported abutments where it is foreseeable that combined effects may lead to unacceptable pile hinging or to brittle failure modes in the piles or connections, superstructure and substructure combined effects should be considered. For integral or semi-integral abutments these connection forces may be neglected.*

4.6.6 Liquefaction of foundation soils

4.6.6.1 Liquefaction potential of foundation soils

Add the following:

The maximum earthquake magnitude for liquefaction analysis shall be based on deaggragation of seismic hazard.

For *Major-route* Bridges: A minimum of seven single-component horizontal ground motion time-histories shall be used. Input ground motion time-histories developed for bridges in the vicinity of the site are permitted with uniform scaling to the site-specific peak horizontal ground acceleration. The mean response quantity shall be used for design.

For *Other* Bridges: A minimum of three single-component horizontal ground motion time-histories shall be used. Input ground motion time-histories developed for bridges in the vicinity of the site are permitted with uniform scaling to the site-specific peak horizontal ground acceleration. The maximum response quantity shall be used for design.

Commentary: *Refer also to Clause 4.4.3.6 for Lifeline bridges. For "Major route" bridges using seven records, this is considered sufficient to adopt the mean response quantities for design. For "Other" bridge using three records, the maximum response quantity is appropriate. If seven records are used as described for the Major route bridges then the average response quantity may be used for Other bridges.*

4.6.8 Fill settlement and approach slabs

Delete the first sentence in the first paragraph and replace with the following:

Approach slabs shall be provided in accordance with Clause 1.7.2.

Commentary: *Project specific design criteria developed by the Ministry may specify settlement slabs (6 m long, measured normal to the abutment) as part of the structural and seismic design criteria. In general approach slabs improve post-seismic performance and vehicle access. For the seismic design of bridges identified in this Supplement under Clause 1.7.2, the role of approach slabs shall emphasize fill settlement. The portion (length) of approach slabs structurally spanning over gaps between end piers or abutments and approach fills shall not be considered as mitigating against fill settlements for post-earthquake bridge access.*

4.7 Concrete structures

4.7.4 Seismic performance category 2

Delete the second sentence and replace with the following: resume

The transverse reinforcement at the top and bottom of a column and in potential plastic hinge zones of beams, columns or piles shall be as specified in Clauses 4.7.5.2.5 and 4.7.5.2.6.

4.7.5 Seismic performance category 3

4.7.5.2 Column requirements

4.7.5.2.3 Flexural resistance

Delete this Sentence.

4.7.5.2.4 Column shear and transverse reinforcement

Replace Clause 4.7.5.2.4 with the following:

The factored shear force, V_f, on each principal axis of each column and concrete pile bent shall be as specified in Clause 4.4.10.4.3.

In lieu of more detailed analysis and design of concrete columns using the commentary below, for columns designed as capacity-protected elements within a ductile substructure, the amount of transverse reinforcement shall not be less than that determined in accordance with Clause 8.9.3, modified by sub-clause (a) below.

The following requirements shall apply to the plastic hinge regions at the top and/or bottom of the column and pile bents:

(a) Shear reinforcement shall be designed in accordance with the requirements of Clause 8.9.3 with $\beta = 0.10$ and $\theta = 45^\circ$. The transverse reinforcement shall consist of hoops, seismic crossties or spirals.

(b) The plastic hinge region shall be assumed to extend down from the soffit of girders or cap beams at the top of columns, and up from the top of foundation at the bottom of columns, a distance taken as the greatest of:
- (i) The maximum cross-sectional dimension of the column;
- (ii) one-sixth of the clear height of the column;
- (iii) 450 mm;
- (iv) The length over which the moment exceeds 80% of the maximum moment.

(c) For tall columns or piers or those having high axial loads, rational analysis that considers potential plastic hinging mechanisms shall be performed to determine the location and extent of plastic hinge regions.

The plastic hinge region at the top of a concrete extended pile bent shall be taken as that specified for columns. 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, taking into account soil-pile interaction, 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.

Commentary:

The amount of transverse reinforcing steel required within plastic hinge regions need not be carried through the remaining length of the columns.

- ***Detailed analysis and design of concrete columns methodology:***

For typical reinforced concrete columns used in bridges in British Columbia, the shear provisions contained in Clause 8.9.3 are unduly conservative and can impede the design of an economic and seismically desirable ductile substructure. In particular, the need for increased column dimensions to meet V_c provisions within 8.9.3 can make it impractical and uneconomic to design capacity-protected footings, pile caps or cap beams.

Acceptable refined seismic shear design methodologies for plastic hinge regions of columns, which takes into account typical bridge column proportions, reinforcing quantities, details and degradation of concrete shear strength is contained in either:

- *Displacement-based Seismic Design of Structures, Priestley, Calvi and Kowalsky, IUSS Press, 2007.*
- *Seismic Design and Retrofit of Bridges, Priestley, Seible and Calvi, Wiley Interscience, 1996.*
- *Bridge Design Practice, Caltrans, 2015 (or latest edition).*

Care is required in the application of equations from references. An implementation example using appropriate resistance factors and material strengths for use with S6-14 is provided below (from Displacement-based Seismic Design of Structures)

$\phi V_{ns} = \phi_c V_c + \phi_{s(} V_s + V_p)$
$V_c = v_c * 0.8\ A_g$

Where

v_c $= \alpha\ \beta\ \lambda\ (f'_c)^{0.5}$
α = (3 – M/(VD)) but no less than 1.0 nor greater than 1.5
β $= 0.5 + 20\ \rho_l$ but no greater than 1.0
λ = factor for degradation in v_c with increasing curvature ductility.
= 0.25 (MPa) for curvature ductilities less than 3
= 0.04 (MPa) for curvature ductilities greater than 15
= varies between using linear interpolation, between curvature ductilities of 3 to 15
= For columns in biaxial bending, similar to above but varying from 0.25 to 0.04 for curvature ductilities between 1 and 13.

$V_s = \pi/(2\ s)\ \{A_v f_{ye}(D - c - c_o)\ \cot(\theta)\}$ (for round columns. For rectangular columns delete $\pi/2$ term and modify A_v as described below))
Where
s = spiral spacing
A_v = Area of reinforcing bar used for spirals (for rectangular columns use total area of all shear bars at the section)
f_y = hoop steel nominal yield stress
D = Column diameter (out to out)
c = depth from extreme compression fibre to neutral axis under the loading considered
c_o = cover to centre of the peripheral spiral cage
Spirals or ties crossed by crack with cot θ measured from vertical, using θ = 35° for design

$V_p = 0.85\ P \tan \alpha$

Where
P = *axial load from bridge weight plus plastic mechanism effects*
α = *angle of inclination of a compression strut through the column, measured from the member's longitudinal axis*

- ***Plastic Hinge Zones in Tall Columns:***

"High axial loads" considers those with greater than 30% of the crush load (f'cAg) of the reinforced concrete section, including axial loads from bridge self weight, any specified live loads to be combined with seismic demands, and from seismic demands. "Tall" columns considers those with clear height to column diameter (H/d), or to least rectangular dimension, greater than 15.

The amount of transverse reinforcing steel required within plastic hinge regions need not be carried through the remaining length of the columns.

4.7.5.2.5 Transverse reinforcement for confinement at plastic hinge regions

Delete phi factors from all equations in this clause.

4.7.5.2.7 Splices

Add the following at the end of the third paragraph:

Welded splices will not be allowed unless consented to by the Ministry.

4.7.5.4 Column connections

Delete the second paragraph and replace with the following:

For lifeline and major route bridges in seismic performance category 3, the design of column connections, including member proportions, details, and reinforcement, shall be based on beam-column joint design methodologies as described in either:

- Seismic Design and Retrofit of Bridges, Priestley and Calvi (1996).
- Caltrans Seismic Design Criteria (latest version, currently 2012)
- ATC-49 Section 8.8.4

Joints shall be designed as capacity-protected elements as described in this Supplement. For bridges in seismic performance category 2, or for "other 'bridges' in seismic performance category 3, in lieu of a detailed beam-column joint design, column transverse reinforcement as specified in Clause 4.7.5.2.5 shall be continued full depth through the joint region.

Commentary: *Rational design of beam-column joints is required for important bridges in high seismic zones. In the absence of an explicit design, other bridges are to have beam-column joints reinforcing extend the full depth of the* *joint. Beam column joints in bridges of SPC 1 should be designed for force transfer as described in Chapter 8 of CHBDC.*

4.7.6 Piles

4.7.6.4 Seismic performance category 3

4.7.6.4.1 General

Add the following paragraph:

For bridges in seismic performance category 3 and where plastic hinging may reasonably be expected to form, concrete piles shall be designed and detailed as ductile components to ensure performance similar to concrete columns designed to Section 4.7.

4.8 Steel structures

4.8.3 Sway stability effects

Add the following:

Commentary: *Guidance on incorporating P-Delta effects can be found in ATC – 32 Clause 3.21.15.*

4.8.4.4.5 Buckling restrained braced frames

Change “R = 5” to “R = 4” at end of the sentence for consistency with Table 4.17.

Commentary: *It is preferable to use analyses that emphasize the deformation demands within the brace when used in bridge applications.*

4.11 Seismic evaluation of existing bridges

Commentary: *The Ministry has established a seismic risk reduction policy for its highway bridges. This policy includes the following initiatives:*

- *Stringent earthquake design standards for planned new bridges.*
- *A program of “seismic retrofitting” to improve the earthquake resistance of existing structures.*

The Ministry has designed bridges to meet modern, evolving earthquake design standards since 1983. These newer bridges may sustain damage but are not expected to collapse in the design earthquake. Structures designed or built prior to 1983, or those having poor seismic detailing or arrangements, are considered potentially vulnerable to collapse or major damage from earthquakes.

In 1989, the Ministry initiated a program of seismic retrofitting to improve the earthquake resistance of existing bridges constructed prior to 1983. The main objectives of the program are as follows:

Minimizing the risks of bridge collapse;

Preserving important highway routes for disaster response and economic recovery after earthquakes;

Reducing damage and minimizing loss of life and injury during and after earthquakes.

A detailed description of the seismic retrofitting program is provided in the report "Bridge Seismic Retrofit Program", BC Ministry of Transportation & Highways, Engineering Branch, February 2000.

Details are contained in the Ministry document, "Seismic Retrofit Design Criteria", June 30, 2005.

Seismic Retrofit Criteria going forward will be based on S6-14 as modified in this Supplement:

S6-14 has made a major shift in the seismic analysis and design of bridges compared to previous codes. It has moved from the use of a force-based design approach with a single level (475 year design event) to a philosophy of performance-based design using multiple earthquake design levels (475, 975 and 2475 year return period events). The Ministry's seismic retrofit criteria, and project-specific seismic criteria adopted beginning also circa 2005, included performance-based and displacement-based requirements and methods.

The Ministry will use the S6-14 performance-based analysis and design approach for evaluation and retrofit of its bridges, as modified within this Supplement. The basic strategy and philosophy behind the Ministry's seismic retrofit program will remain unchanged.

Sections 4.11 and 4.12 in this Supplement, which build on provisions elsewhere in Chapter 4 and this Supplement, provide the Ministry's general requirements for analysis and design of seismic retrofits that will be used going forward.

4.11.1 General

Add the following paragraph:

Existing bridges will be evaluated based on performance-based principles and criteria from Clause 4.4.6.3 based on hazard levels designated by the Ministry. Seismic evaluations shall assess the expected performance of the bridge at the required hazard levels.

4.11.3 Seismic Hazard

Unless otherwise specified by the Ministry, the hazard having a 2% in 50 year probability of exceedance shall be used for seismic evaluation.

Commentary: *The previous baseline hazard for seismic evaluation of existing bridges was a 10% in 50 year probability of exceedance. The Ministry's objective is to assess and retrofit those bridges in its Seismic Retrofit program that are expected to have remaining economic lives in excess of 20 years following renewal or retrofit, to at least a collapse prevention state for a hazard having a 2% in 50 year probability of exceedance. For bridges expected to have shorter functional lives, but which are targeted for seismic retrofit, then a hazard not lower than 10% in 50 years shall be specified.*

4.11.4 Performance criteria for performance-based design approach

The service and damage criteria for sites without liquefaction (site class A to E), shall be in accordance to Table 4.15, unless otherwise specified or consented to by the Ministry.

The service and damage criteria for sites with liquefaction (site class F), shall be specified by the Ministry on a project by project basis. The performance criteria as a minimum shall be “Life Safety” and “Probable Replacement”.

Commentary: *The previous performance criteria for bridge seismic retrofit prior to S6-14 was determined using a staged approach. This staged approach will be used going forward as well. In the current stage, the objective will be to continue to reduce the risk of bridge collapse. The ultimate objective is to work towards achieving performance criteria equivalent to new bridges using a staged approach. There may be aspects of existing bridges that preclude economical achievement of the ultimate objective.*

4.11.5 Performance criteria for force-based design approach

Clauses 4.11.5.1 and 4.11.5.2 will not be used for seismic assessment or retrofit of Ministry bridges.

Commentary: *This does not preclude the reliance on elastic component strengths having adequate reserve margin as a lateral-load resisting mechanism in existing bridges. Evaluations shall use displacement-based or time-history methods wherever practicable. The latter may be applicable to base isolation or added damping strategies. For screening-level evaluation of bridges as part of seismic retrofit planning and prioritizing, elastic methods may be appropriate. Elastic methods shall not be sufficient analyses for decisions related to renewal / retrofit versus replacement, or as meeting the requirements for seismic assessment to this Supplement.*

4.11.6 Load factors and load combinations for seismic evaluation

Add sentences to end of Clause:

The assessment of biaxial effects on failure modes shall be addressed explicitly in the evaluation of existing bridges.

Commentary: *Biaxial bending in poorly detailed, brittle components may lead to spalling, loss of structural integrity of the core of the member and potential collapse. Evaluation for the potential for these failure modes is therefore essential in existing bridges, and retrofit measures considered must also address this potential.*

4.11.7 Minimum support length

Replace last sentence with:

Alternately, longitudinal restrainers complying with Clause 4.4.10.6 shall be provided, or structurally integral superstructure to sub-structure connections having sufficient capacity to be capacity-protected elements may be relied on.

4.11.9 Required response modification factor for force-based design approach

Delete clause.

4.11.10 Response modification factor for existing substructure elements

Response modification factors shall not be used in lieu of explicit displacement-based methods.

Delete the words “modification factors” from sub-clause (a).

Commentary: *Sub-clauses (a) and (b) as modified above remain applicable.*

4.11.11 Evaluation acceptance criteria

Delete second paragraph.

4.11.12 Bridge access

Modify sentence by deleting “… for Major-Route bridges located in Seismic performance category 3.”

Commentary: *Damage to embankments and abutments shall be evaluated.*

4.11.13 Liquefaction of foundation soils

Delete first sentence and replace with “The potential for liquefaction of the foundation soils shall be evaluated as required to determine performance.”

Delete sub-clauses (a) and (b).

4.11.14 Soil-structure interaction

Delete entire Clause and replace with the following Clause

Soil-foundation-structure interaction shall be assessed in accordance with Clause 4.6.4

4.11.15 Seismic Evaluation Report

A Structure Seismic Evaluation Report shall be prepared for Ministry review and acceptance. The report will incorporate findings from a Detailed Condition Assessment Report and a Structure Evaluation Report, when provided or created prior to the creation of the Structure Seismic Evaluation Report. The Structure Seismic Evaluation Report is intended to define all of the vulnerabilities for the existing structure and to provide recommendations and cost estimates for seismic retrofit actions to achieve the performance objectives for the site and classification and shall contain the following as a minimum:

- The specified performance objectives.
- A summary of design response spectra and, where applicable, ground motion time histories.
- Desktop assessment of liquefaction at the site for the hazards having a 10% and a 2% in 50 year probability of exceedance.
- Description of methodology and parameters for structural and geotechnical assessment.
- Procedures for establishing material properties and design/constructed details, and the methodology used for determining ductility demands and capacities of existing structural components/connections.
- Define reference materials used and all assumptions made as part of this work. Provide recommendations for any additional field and/or desktop work to verify or alter them.
- Identification and prioritization, based on expected performance, of seismically deficient areas of the structure and foundations.
- Description of the current seismic load paths through the structure [Load Patch Capacity Assessment], key components, their criticality, behaviour, reliability and their assessed seismic performance.
- Summary of the displacement demands and capacities from the analysis of the current structure.
- Discussion of high demand vulnerable components, for the current structure, that could affect use and expected damage, the nature of the associated short term actions and time to restore service, the type of restored service [emergency vehicle access lane in each direction only, full access with load limits, full access] and the stabilization work and/or full repair work, if

applicable, to restore the structure to its pre-event service level.

- Description of recommended conceptual retrofit measures, their capacity improvement ratio, including schematic sketches, quantities, cost estimates, and appropriate back-up data to achieve performance measures.

- Discussion of high demand vulnerable components, for the retrofitted structure, that could affect use and expected damage, the nature of the associated short term actions and time to restore service, the type of restored service [emergency vehicle access lane in each direction only, full access with load limits, full access] and the stabilization work and/or full repair work, if applicable, to restore the structure to its pre-event service level.

4.12 Seismic Rehabilitation

4.12.1 Performance criteria

Delete entire clause and replace with:

4.12.1 General

Performance-based design will be required for all seismic rehabilitation (retrofit) of bridges of all importance classifications and performance categories. The Ministry will designate the importance classification.

The level and type of retrofit to be implemented shall consider the existing seismic resistance of the bridge and the type of modifications to the structure and substructure that will allow the bridge to meet the performance objectives specified by the Ministry. Analytical studies shall be carried out and experimental studies may be used to determine retrofit alternatives for the bridge.

Commentary: *The performance levels, type and staging of seismic retrofit to be implemented shall consider:*

- *The seismic hazard of the bridge location.*

- *The importance of the bridge to the transportation network considering post-disaster response and recovery, and longer term local and regional economic recovery.*

- *The existing seismic resistance and resilience of the bridge, considering the bridge form and materials, and the severity and consequences of assessed seismic vulnerabilities.*

- *The expected remaining in-service life of the bridge.*

- *The nature and timing of other bridge renewal measures identified and planned.*

- *The costs and benefits of implementing seismic upgrades.*

- *The type of modifications to the structure, substructure and foundation soils that will allow the bridge to meet the performance objectives specified by the Ministry.*

Analytical studies for the structure and soils to demonstrate performance using deformations shall be carried out for the design of seismic upgrades. Experimental studies may be used to aid in the assessment and design of retrofit alternatives or works for the bridge. Material testing shall be done where appropriate to either assess the bridge performance or to design upgrading works.

Commentary: *The goal of Clause 4.12 is to identify and implement a cost-effective seismic upgrading strategy that meets the prescribed performance requirements and which can be integrated into other renewal works planned for each bridge. The Ministry will specify objectives, requirements and implementation staging in project-specific Seismic Criteria. Principles to guide the seismic upgrading strategy include:*

- *The assessment of seismic vulnerabilities and design of upgrading works shall use displacement-based methods wherever applicable. Elastic demands and designs may be unavoidable for some existing bridges, but where used shall provide the performance requirements with an appropriate margin of reserve strength. Force reductions based on ductility factors as in a force-based design approach shall not be used.*

- *Where appropriate, the Province will assess and target retrofit levels for existing bridges to a 2% in 50 year performance level. The Ministry's seismic upgrading program started in the 1990's targeted collapse prevention for a seismic hazard having a 10% probability of exceedance in 50 years. For critical bridges post-seismic usage by emergency vehicles was also specified. Currently, all major Lower Mainland Lifeline Bridges have been upgraded or replaced (or slated for replacement), and many other important bridges have been assessed and seismically upgraded. The remaining older, deficient bridges not yet upgraded have since expended an additional two decades of their remaining service lives. As such, the remaining economic lives of some of these older bridges will be significantly shorter than the life of a new bridge. Where economically feasible, retrofit to a 2% in 50 year level should be adopted for bridges reasonably expected to remain in service for 20 years or more. These may include large or important bridges which are expensive to replace, bridges having their economic lifespans extended through renewal measures, or other bridges designated by the Ministry. For other bridges in the retrofit program, seismic hazard levels and performance requirements used previously, general a collapse prevention or risk mitigation to a 10% hazard level, may provide appropriate levels of protection.*

- *Given the limited economic lives of some existing bridges in the retrofit program, seismic upgrades to be implemented are likely to be the best or sole opportunity to upgrade these bridges. The retrofit level to be implemented should in general therefore be implemented as a single-stage retrofit, although more than one contract package may be adopted.*

- *For bridges to be renewed to extend their economical lives potentially beyond approximately 20 years, seismic assessments shall be performed and vulnerabilities shall be identified through analysis and assessment for a 2% in 50 year hazard. Sufficient information including analysis, assessment and retrofit strategy should be completed, based on analyses and methods outlined in this Supplement, such that an informed decision can be made regarding renewal or replacement of the bridge. For any bridge for which a seismic retrofit is contemplated, other than for an initial screening of an inventory of bridges, a displacement-based performance assessment using static pushover models shall be used. Where substructures are found to remain essentially elastic, and whose capacities would not be exceeded, a push-over assessment becomes moot.*

Unless otherwise specified by the Ministry, the minimum performance levels to be used for seismic rehabilitation shall be in accordance with Section 4.11.4.

Add the following:

4.12.5 Seismic retrofit strategy report

A Seismic Retrofit Strategy Report shall be prepared for Ministry review and acceptance. The Seismic Retrofit Strategy Report shall contain the following as a minimum:

- Project-specific seismic retrofit design criteria.
- A summary of design response spectra and, where applicable, ground motion time histories.
- Description of methodology and parameters for structural and geotechnical modelling, analysis and design.
- Procedures for establishing properties of existing materials and the methodology used for determining capacities of existing structural components.
- Description of the seismic load path through the structure, key components, their importance and behaviour and their assessed seismic performance.
- Summary of the results and demands from the analysis.
- Identification and prioritization of seismically deficient areas of the structure, including geotechnical deficiencies.

- Description of conceptual retrofit measures and their design philosophies including preliminary drawings, estimated costs, appropriate back-up data, and aesthetic considerations.
- Discussion of expected damage and the nature of the repairs anticipated, if applicable, to restore the structure, under traffic as required, to the specified service level.
- Summary of the recommended retrofit scheme to proceed with in the detailed design phase.
- Discussion of the long-term reliability and required maintenance of the proposed retrofit measures.
- All summary testhole/testpit logs.

The report shall be submitted for ministry review prior to undertaking the detailed design. It shall be updated to include any modifications made as a result of the ministry review. A final version of the report shall also be provided after construction to include any modifications resulting from the construction work.