4.2 Definitions

Add the following:

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.*

Capacity design involves an explicit selection of a plastic mechanism as the lateral earthquake resisting system. This system includes 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. 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.

Probable resistance**:** *The probable resistance is larger than the expected nominal resistance and represents the increased resistance that ductile substructure element (plastic hinges) can develop accounting for effects such as rebar strain hardening, concrete confinement, expected material properties, etc. Adjacent capacity protected members and/or undesirable failure modes are designed for the force effect resulting from the plastic hinges (structural fuses) attaining their probable resistance.*

Static Pushover analysis *- Pushover analyses are used to determine capacity design demands and to assess structural behaviour and damage at each stage of inelastic deformation of the lateral load resisting system. Section capacities account for degradation with increasing ductility demands. Local deformations and strains from the analysis allow for damage and performance assessments at all specified earthquake levels, demonstrating whether the design performance criteria have been met.*

Add the following definitions:

Analytical Plastic Hinge Length: the calculated equivalent length of a plastic hinge region used for analytical purposes, and over which the plastic curvature is assumed constant for estimating plastic rotation and plastic curvature.

Earthquake-Resisting System (ERS): a system that provides sufficient strength, ductility or energy dissipation for the bridge, ensures a load path for gravity loads, and controls seismic performance of the bridge.

Earthquake-Resisting Element (ERE): bridge elements within the ERS that transfers lateral loads, undergoes inelastic deformation, dissipates energy, or increases bridge damping.

Extended pile bent: Replace the definition with the following: 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. Also includes Type I and Type II shafts. (Note: Where “pile bent” is used in this Section, it may be interpreted as an extended pile bent.)

Expected nominal resistance: the resistance based on expected material properties, calculated with the material resistance factors taken as 1.0.

Impedance: the product of V_s and mass density.

Impedance Ratio: the impedance of the material above the impedance contrast divided by the impedance of the material below the impedance contrast.

Plastic Hinge: a region of a structural member that undergoes flexural yielding and plastic rotation while retaining flexural strength.

Plastic Hinge Region/Zone: a region/zone of a structural member expected to, or with the potential to, form a plastic hinge and that therefore requires special seismic detailing to provide ductility.

Type I Pile Shaft: a drilled or driven shaft foundation having the same confined core diameter as that of the supported column but may have the same or different concrete cover and area of transverse and longitudinal reinforcement as the supported column. As defined by Caltrans Seismic Design Criteria, Version 2.0, April 2019.

Type II Pile Shaft: a drilled or driven shaft foundation that is larger in diameter than the supported column and has a reinforcing cage larger than and independent of columns. As defined by Caltrans Seismic Design Criteria, Version 2.0, April 2019.

Commentary:

To ensure ductile behaviour, S6-19 and the Supplement to S6-19 require enhanced seismic detailing for plastic hinge regions/zones and do not permit lap splices in these regions/zones.

The analytical plastic hinge length is used for calculating the plastic rotation and plastic curvature. The analytical plastic hinge length is different than plastic hinge region/zone. The AASHTO Guide Specification for LRFD Seismic Bridge Design (2nd edition) or the Caltrans Seismic Design Criteria Ver 2 may be used for calculating the analytical plastic hinge length.

4.3 Abbreviation and symbols

4.3.2 Symbols

Add the following:

$PGA(X)$ = peak ground acceleration, expressed as a ratio to gravitational acceleration, for seismic site designation X.

$PGV(X)$ = peak ground velocity, in m/s, for seismic site designation X.

$S_a(T, X)$ = 5%-damped spectral acceleration, expressed as a ratio to gravitational acceleration, at period T for seismic site designation X.

X = seismic site designation, either X_V or X_S.

X_S = seismic site designation in terms of Site Class, where S is the Site Class determined in accordance with Clause 4.4.3.2.(2).

X_V = seismic site designation in terms of $\bar{V}_{s30}$, where V is the $\bar{V}_{s30}$ value calculated from in situ measurements of shear wave velocity.

ε_{ye} = strain corresponding to the expected yield strength of structural steel.

ε_{sh} = strain at the onset of strain hardening of structural steel.

ε_{ue} = strain corresponding to the expected tensile strength of structural steel.

Commentary:

Factored loads (eg. P_f, V_f) are replaced by the design forces in accordance with code provisions when using capacity design approach.

The expected tensile strength of structural steel, ε_{sh} and ε_{ue} should be based on mean values of tested material properties.

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 designated by the Ministry.

Seismic design should be considered on a case-by-case basis for temporary bridges planned to be in place for more than two years.

For temporary Lifeline and Major-Route bridges, a return period of at least 100 years should be considered.

Seismic design should be considered for all partially constructed bridges that carry or cross over public vehicular traffic, rail lines or navigable waters when the duration of construction is expected to exceed two years. For Lifeline and Major-Route bridges, a return period of at least 100 years should be considered.

4.4.3 Seismic hazard

4.4.3.1 General

Replace the first paragraph with the following:

The 5% damped horizontal spectral response acceleration values, $S_a(T, X)$, in units of g and where T is in seconds, for seismic site designation X, are established for the National Building Code of Canada (NBCC). These values shall be determined from NRCan 2020 National Building Code of Canada seismic hazard tool (https://www.seismescanada.rncan.gc.ca/hazard-alea/interpolat/nbc2020-cnb2020-en.php) for the required probability of exceedance in 50 years.

Replace the second paragraph with the following:

The peak ground acceleration, *PGA(X)*, peak ground velocity, *PGV(X)*, and the 5% damped spectral response acceleration values, $S_a(T, X)$ for seismic site designation *X* for periods *T* of 0.2 s, 0.5 s, 1.0 s, 2.0 s, 5.0 s, and 10.0 s shall be determined for the hazard levels specified in Clause 4.4.6.2 for performance-based design, and for 2% in 50 years (2475-year return period) hazard level for force-based design.

Replace the fourth paragraph with the following:

For structures on all seismic site designations other than X_F, site-specific response analysis may also be conducted to obtain design spectral values when Consented to by the Ministry.

For all Lifeline bridges and Major-Route bridges in Seismic Performance Category 3, where the period of vibration of the elastic soil columns at foundation locations is within 0.5 to 2 times the period of vibration of the structure, the design response spectrum may be established using 1-D or 2-D site-specific response analysis if any of the following conditions are met, as agreed or directed by the Ministry:

- Non-gradational sites where shear wave velocity does not increase gradually with depth to a depth of at least 30 m, or
- High impedance contrast sites where there is an abrupt change of impedance over a few meters, with an impedance ratio greater than 4 or less than 0.25.

For structures on seismic site designation X_F with liquefiable soils:

- The requirements of Clause 6.14.8 shall be met.
- For structures with a fundamental period of vibration equal to or less than 0.5 s that are built on liquefiable soils, the seismic site designation, *X* and the corresponding values of $S_a(T, X)$ and *PGA(X)* may be determined by assuming that the soils are not liquefiable.
- For Major-Route bridges in Seismic Performance Category 1 and 2, and for all Other bridges, the design response spectrum shall be the code-based design response spectrum using non-liquefied soil properties.
- For all Lifeline bridges and for Major-Route bridges in Seismic Performance Category 3, the design response spectrum shall be established using 1-D or 2-D site-specific response analysis.

For structures on seismic site designation X_F without liquefiable soils:

- For Major-Route and Other bridges in Seismic Performance Category 2 and Other bridges in Seismic Performance Category 3, the design response spectrum is the envelope of the code-based design response spectra for seismic site designation X_E for $T \leq 0.8$ sec and seismic site designation X_V or X_E for $T \geq 0.8$ sec.
- For all Lifeline bridges and Major-Route bridges in Seismic Performance Category 3, the design response spectrum shall be established using 1-D or 2-D site-specific response analysis.

When the design response spectrum is established using 1-D or 2-D site-specific response analysis, a high degree of site understanding as defined in Clause 6.5.3 is required.

For all seismic site designations, the design response spectra from site-specific response analysis shall not be less than 80% of the code-based spectra for the applicable seismic site designation. For liquefiable soil, non-liquefied soil properties shall be used when establishing the code-based design spectra.

Commentary:

In National Building Code of Canada 2020, the 6th generation seismic hazard model is adopted along with new seismic site designation definition. All references to Ste Class are now changed to seismic site designation X_V or X_S.

For projects in the regions where hydraulic fracturing (fracking) or deep wastewater disposal are planned or have been carried out, site-specific hazard analysis should be conducted to include injection-induced earthquakes as part of the seismic design.

For site-specific response analysis, the seismic site designation X_F is categorized into two groups:

1. *Liquefiable soil that is horizontally continuous across the entire footprint of an abutment, pier, or geotechnical system, and*
2. *Other seismic site designation X_F soils exhibiting ground profile characteristics of Site Class F as outlined in Table 4.1-B, except for the final criterion that pertains to liquefiable soils.*

The analysis requirements for seismic site designation X_F soils are addressed in clause 6.14.

Design response spectra from site-specific response analysis should have a horizontal plateau for short period ranges. The design response spectrum should not increase with an increase in period.

For sites where the soil deposition is not horizontal, 2-D site-specific response analysis is more appropriate than 1-D site-specific response analysis.

Spectral response acceleration values from the NBCC 2020 hazard tool are developed for gradational sites.

4.4.3.2 Site properties

Replace this clause with the following:

1) Except as provided in Sentence 2), the seismic site designation shall be X_V determined using the time-weighted average shear wave velocity in top 30 m, $\bar{V}_{s30}$, calculated from in-situ measurements of shear wave velocity, as follows:
   a) The site-specific shear wave velocity profile of the ground shall be directly measured to at least 30 m below ground surface, or to at least 5 m depth below a sound rock layer.
   b) The time-weighted average shear wave velocity in top 30 m, $\bar{V}_{s30}$, shall be calculated as

$$\bar{V}_{s30} = \frac{30}{\Sigma[\, h_i / V_{s,i} \,]}$$

where $V_{s,i}$ is in units of m/s, h_i in units of meters, and Σh_i = 30 m.

c) For the ground profiles described in Table 4.1-A, the site designation shall be determined in accordance with the Table, and

d) For all other ground profiles, the seismic site designation shall be X_V, where V is the value of $\bar{V}_{s30}$ as defined in this clause.

2) Where $\bar{V}_{s30}$ calculated from direct in-situ measurements is not available, the seismic site designation shall be X_S, where S is the Site Class determined in accordance with Table 4.1-B.

Table 4.1-A
Exceptions for Site Designation Using $\bar{V}_{s30}$ Calculated from In-Situ Measurements
(See Clause 4.4.3.2.(1))

Grond Profile Characteristics		Site Designation
Average Shear Wave Velocity in Top 30 m, $\bar{V}_{s30}$, Calculated from In-Situ Measurements, in m/s	Additional Characteristics	
$\bar{V}_{s30} > 760$	Ground profile contains more than 3 m of softer materials between rock and the underside of footing or mat foundations	X_{760}
$\bar{V}_{s30} > 140$	Ground profile contains a cumulative thickness more than 3 m of soil with all the following characteristics: • Plasticity Index, $PI > 20$, • Moisture content, $w \geq 40\%$, and • Undrained shear strength, $\bar{s}_u < 25$ kPa	X_E
$\bar{V}_{s30} > 140$	Ground profile with any of the following characteristics: • Quick and highly sensitive silts and clay ($S_t > 8$), • Peat, organic silts and/or organic clays with a cumulative thickness greater than 3 m, • High-plastic clays ($PI > 75$) more than 8 m thick, • Soft to firm ($\bar{s}_u < 50$ kPa) silts and/or clays with a cumulative thickness more than 30 m, • Municipal solid waste and/or other landfill materials, or • A horizontally continuous liquefiable layer across the entire footprint of an abutment, pier or geotechnical system.	X_F
$\bar{V}_{s30} \leq 140$	N/A	X_F

Table 4.1-B
Site Classes, S, for Seismic Site Designation X_S
(See Clause 4.4.3.2.(2))

<table>
<tr><th rowspan="2">Site Class, S</th><th rowspan="2">Ground Profile</th><th colspan="3">Ground Profile Characteristics</th></tr>
<tr><th>Average Shear Wave Velocity in Top 30 m, $\overline{V}_{s30}$, in m/s</th><th>Average Standard Penetration Resistance in Top 30 m, $\overline{N}_{60}$, in Blows per 0.3 m</th><th>Average Undrained Shear Strength in Top 30 m, $\bar{s}_u$, in kPa</th></tr>
<tr><td>A</td><td>Hard rock</td><td>$\overline{V}_{s30} > 1500$</td><td>Not applicable</td><td>Not applicable</td></tr>
<tr><td>B</td><td>Rock</td><td>$760 < \overline{V}_{s30} \le 1500$</td><td>Not applicable</td><td>Not applicable</td></tr>
<tr><td>C</td><td>Very dense soil and soft rock</td><td>$360 < \overline{V}_{s30} \le 760$</td><td>$\overline{N}_{60} > 50$</td><td>$\bar{s}_u > 100$</td></tr>
<tr><td>D</td><td>Stiff soil</td><td>$180 < \overline{V}_{s30} \le 360$</td><td>$15 < \overline{N}_{60} \le 50$</td><td>$50 < \bar{s}_u \le 100$</td></tr>
<tr><td rowspan="2">E</td><td rowspan="2">Soft soil</td><td>$140 < \overline{V}_{s30} \le 180$</td><td>$10 < \overline{N}_{60} \le 15$</td><td>$40 < \bar{s}_u \le 50$</td></tr>
<tr><td colspan="3">Any ground profile other than Site Class F that contains a cumulative thickness more than 3 m of soil with all the following characteristics:
• Plasticity Index, $PI > 20$,
• Moisture content, $w \ge 40\%$, and
• Undrained shear strength, $\bar{s}_u < 25$ kPa</td></tr>
<tr><td rowspan="2">F</td><td rowspan="2">Other soils</td><td>$\overline{V}_{s30} \le 140$</td><td>$\overline{N}_{60} \le 10$</td><td>$\bar{s}_u \le 40$</td></tr>
<tr><td colspan="3">Ground profile with any of the following characteristics:
• Quick and highly sensitive silts and clay ($S_t > 8$),
• Peat, organic silts and/or organic clays with a cumulative thickness greater than 3 m,
• High-plastic clays ($PI > 75$) more than 8 m thick,
• Soft to firm ($\bar{s}_u < 50$ kPa) silts and/or clays with a cumulative thickness more than 30 m,
• Municipal solid waste and/or other landfill materials, or
• A horizontally continuous liquefiable layer across the entire footprint of an abutment, pier or geotechnical system.</td></tr>
</table>

***Commentary**: It is preferable to determine the seismic site designation as X_V. This typically results in a lower design response spectrum than a seismic site designation X_S based on Site Class.*

4.4.3.3 Site coefficients

Delete this clause.

***Commentary**: Site coefficients are no longer used as the spectral response accelerations from the 6th generation seismic hazard tool (NBCC 2020) already account for site properties using site designations.*

4.4.3.4 Design response spectral (acceleration and displacement)

Replace this clause with the following:

The design spectral acceleration, $S(T)$ shall be determined as follows, using log-log interpolation for intermediate values of T:

$S(T) = S_a(0.2, X)$ or $S_a(0.5, X)$, whichever is greater for $T \leq 0.2$ s

$= S_a(0.5, X)$ for $T = 0.5$ s

$= S_a(1.0, X)$ for $T = 1.0$ s

$= S_a(2.0, X)$ for $T = 2.0$ s

$= S_a(5.0, X)$ for $T = 5.0$ s

$= S_a(10.0, X)$ for $T = 10.0$ s

The design spectral displacement values $S_d(T)$ at periods T= 0.0, 0.2, 0.5, 1.0, 2.0, 5.0, and 10.0 s shall be determined using $S_d(T) = 250\, S(T)\, T^2$ (in millimeters). Values for intermediate values of T shall be determined using log-log interpolation.

Commentary:

The value of $S(T)$ for $T_i < T < T_j$ can be determined using log-log interpolation as follows:

$$S(T) = 10^{\log(S(T_i)) + \frac{\log(T) - \log(T_i)}{\log(T_j) - \log(T_i)}\left[\log(S(T_j)) - \log(S(T_i))\right]}$$

When interpolating for the intermediate values of design spectral accelerations and displacements, log-log interpolation significantly improves the accuracy of seismic demands compared to linear interpolation.

4.4.3.6 Input ground motion records for time history analyses

Add the following to the first paragraph:

If Consented to by the Ministry, two or more site-specific target response spectra may be allowed. Input ground records shall have similar spectral shape to the target response spectrum.

Delete the second paragraph and replace with:

Eleven or more ground motion records shall be used in design for each target spectra. When only one suite of ground motion records is used in the analysis, the mean response quantity shall be used in design. When two or more suites of 11 or more ground motion records are used, the design seismic demand shall be taken as the largest of the mean values of each suite.

Add the following to the end of the third paragraph:

Vertical ground motions may be determined by carrying out an independent hazard calculation using vertical ground motion prediction equations to develop a vertical response spectrum. The methodology and ground motions shall be Consented to by the Ministry.

Add the following paragraph:

Deviations to the selection and use of ground motions shall be Consented to by the Ministry.

Commentary*: The Commentary on CSA S6:19, Canadian Highway Bridge Design Code should be used for guidance and additional references for the selection and use of ground motions.*

4.4.5.2 Single-span bridges

4.4.5.2.1 Analysis requirements

Add the following:

Single-span bridges with seismic site designation X_F shall be designed using PBD as specified for multi-span bridges in Clause 4.4.5.3.

In the restrained directions, the substructure of all single-span bridges shall be designed for the connection force effect from Clause 4.4.10.1, or for the seismic design forces obtained from EDA or ESA.

Commentary: *Continuous and reliable load path to transfer all seismic inertial loads from point of application to surrounding soils is essential for earthquake resistance systems. The substructure design of single-span bridges is only needed in the restrained directions between the superstructure and the substructure.*

4.4.5.3 Multi-span bridges

4.4.5.3.1 Analysis requirements and design approach

Add the following note to Table 4.11:

Bridges which may be subjected to the effects of liquefaction shall be designed using performance-based design.

Add the following note to Table 4.12:

EDA and ISPA are required for bridges which are subjected to the effects of liquefaction.

Add the following note to Table 4.13:

For PBD, ISPA is required unless the structure behaves in an essentially elastic manner.

Add the following:

Free-field response spectrum for ESA, EDA, and ISPA or free-field time history ground motions for NTHA shall be established using either code specified values or site response analysis. Free-field response spectrum or free-field time history ground motions may be computed at an elevation determined jointly by the structural and geotechnical engineer and Consented to by the Ministry.

Commentary: *The depth-to-fixity may be used for determining the foundation input motion. The depth-to-fixity may be derived by equating the lateral stiffness of the cantilever to that of the elastic soil-pile system (Chai 2002). The depth-to-maximum-moment is different than the depth-to-fixity.*

Add the following:

As a minimum, the following geotechnical engineering input shall be incorporated in the structural analysis methods described in Table 4.12 and Table 4.13:

- 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.
- 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 6.14.5 for shallow foundations, Clause 6.14.6 for deep foundations, and Clause 6.14.7 for abutment and approach fill interaction.
- Where applicable (e.g., liquefaction-induced lateral spreading or settlements), the effects of kinematic loading from inelastic ground deformations on the displacement and other effects of inertial loading shall be evaluated and combined using the combinations described below:
    - 100% inertial demands obtained from Clause 6.14.8.2.2, and
    - 100% kinematic demands ±50% inertial demands, combined in accordance with Clause 6.14.8.3 and this Supplement.
- Nonlinear time history analysis (NTHA): These analyses shall be carried out on a full or partial model of the bridge system incorporating the nonlinear behaviour of foundation soils and foundation elements. For NTHA, explicit pile foundation modelling is required. Computer software used for this purpose shall have the capability to incorporate nonlinear soil effects, pre-and post-earthquake stress-strain-strength characteristics of soils, and nonlinear structure effects. These analyses shall be either 2-D or 3-D. Unless otherwise specified by the Ministry, analyses shall be carried out for all input ground motions defined in Clause 4.4.3.6.
- A range of possible soil stiffness shall be evaluated for ESA, EDA, ISPA, and NTHA analysis based on accepted geotechnical methods using soil parameters based on field and laboratory testing. A study shall be made on the sensitivity of bridge seismic response to the variation in the soil stiffness.

Commentary*:*

Tables 4.12 and 4.13 apply to structural analyses including appropriate modelling for important soil-structure interaction effects in all analysis types.

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

Project specific seismic specifications will be provided by the Ministry for Lifeline, Major-Route bridges, or other specific projects when needed.

Add the following clause:

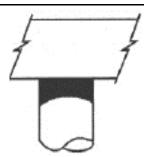
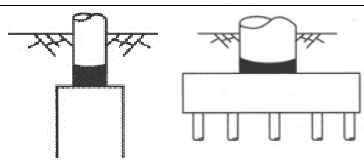
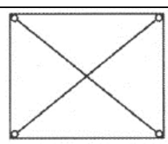
4.4.5.4 Earthquake Resisting System (ERS) and Earthquake Resisting Element (ERE)

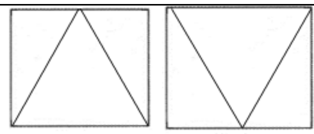
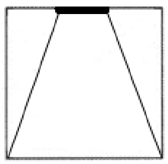
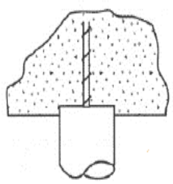
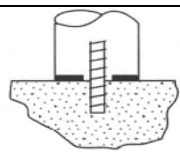
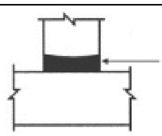
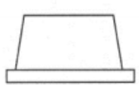
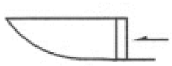
For Seismic Performance Category 2 and 3, all bridges and their foundations shall have a clearly identifiable ERS and ERE to achieve the seismic design requirements. The ERS shall be designed to ensure a continuous and reliable load path to transfer all seismic inertial loads from point of application into the surrounding soil.

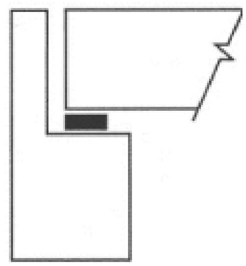
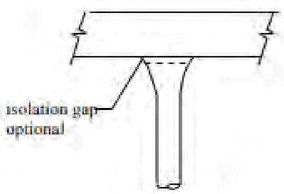
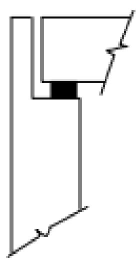
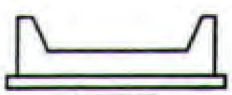
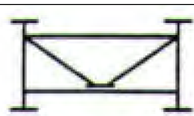
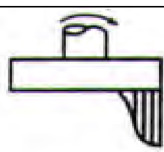
Limitations on the use of EREs are provided in Table 4.4.5.4-1. EREs are categorized into Permissible, Not Permissible, and Permissible when Consented to by the Ministry.

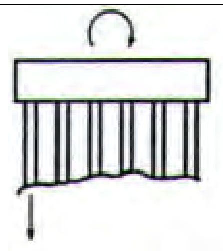
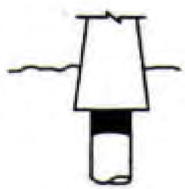
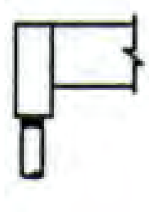
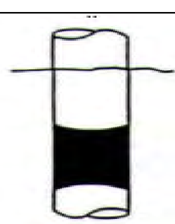
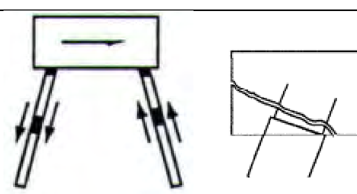
Use of EREs not identified in Table 4.4.5.4-1, require Consent by the Ministry. Where practical, preference shall be given to ERSs and EREs that promote low-damage and high-resilience performance.

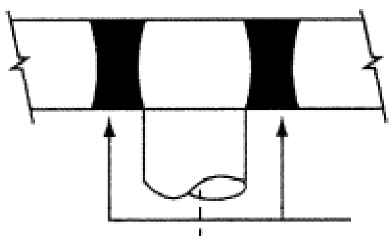
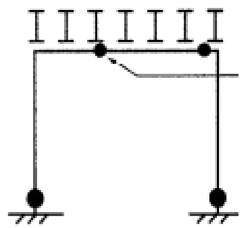
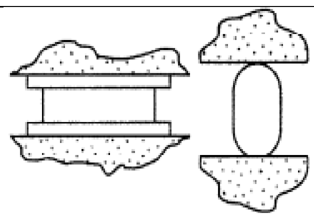
Table 4.4.5.4-1: Earthquake-Resisting Elements (EREs)

Item	Illustration	Description	Conditions of Use
1		Column plastic hinges below cap beams, including extended pile bents.	Permissible
2		Near surface and at surface column plastic hinges above capacity-protected foundations, including spread footings and pile caps.	Permissible
3		Seismic isolation bearings or bearing designs to accommodate total seismic displacement.	Permissible
4		Ductile concentrically braced frames.	Permissible in the substructure.

5		Chevron-braced and V-braced systems	Permissible in the substructure.
6		Ductile eccentrically braced frame.	Permissible in the substructure when Consented to by the Ministry and only when there are no girders supported on the cap beam that has the shear link
7		Piles with ‘pinned-head’ conditions	Permissible when Consented to by the Ministry
8		Columns with moment reducing or pinned hinge details	Permissible in substructure when Consented to by the Ministry
9		Plastic hinges at base of wall piers and rectangular columns in weak direction	Permissible
10		Pier walls with or without piles	Permissible
11		Spread footings meeting the overturning criteria of Clause 6.14	Permissible
12		Passive abutment resistance required as part of ERS. Passive abutment resistance shall be based on 70% of ultimate capacity as determined in accordance with Clause 6.14.7.1.	Permissible

13		Seat abutments whose backwall is designed to fuse.	Permissible
14		Columns with architectural flares – with or without an isolation gap As per the requirements of CALTRANS SEISMIC DESIGN CRITERIA VERSION 2.0 APRIL 2019 Clause 7.6.3 and the AASHTO Guide Specification for LRFD Seismic Design, 2011 Section 8.14 for isolation gaps.	Permissible when Consented to by the Ministry.
15		Seat abutments whose backwall is designed to resist the expected impact force in an essentially elastic manner	Permissible
16		Sliding of spread footing abutment allowed to limit force transfer.	Permissible when Consented to by the Ministry
17		Ductile End-diaphragms in superstructure.	Permissible when Consented to by the Ministry
18		Foundations permitted to rock Use rocking criteria according to AASHTO Guide Specification for LRFD Seismic Design, 2011 Appendix A.	Permissible when Consented to by the Ministry.

		NTHA with comprehensive sensitivity analysis of foundation, structure, and ground motion assumptions required.	
19		More than the outer line of piles in group systems allowed to plunge or uplift under seismic loadings.	Permissible when Consented to by the Ministry.
20		Wall piers on pile foundations that are not strong enough to force plastic hinging into the wall, and are not designed for the seismic elastic forces	Permissible when Consented to by the Ministry.
21		Plumb piles that are not capacity-protected for integral abutment piles, semi-integral abutment piles or pile-supported seat abutments that are not fused transversely	Permissible when Consented to by the Ministry.
22		In-ground hinging in Type I or II pile shafts.	Permissible when Consented to by the Ministry.
23		Batter pile systems in which the geotechnical capacities and/or in-ground hinging define the plastic mechanisms.	Not permissible.

24		Plastic hinges in the superstructure	Not permissible.
25		Frame with cap beam plastic hinging	Not permissible.
26		Bearing systems that do not provide for the expected displacements and/or forces (e.g., rocker bearings)	Not permissible.

Item 2: For Type II pile shafts, the shaft shall be designed as a capacity-protected element and column longitudinal reinforcement shall be embedded into oversized shafts per Caltrans Seismic Design Criteria Version 2.0, April 2019, clause 8.3.2. Sensitivity analysis, using appropriate bounds on the soil-pile interaction model, shall be performed on shear and moment demands.

Items 2, 20, 21 and 22:

In-ground plastic hinges are only permissible if reasonably accessible for inspection and repair. In-ground plastic hinge locations shall be Consented to by the Ministry.

Deep in-ground plastic hinge locations are only permissible for liquefaction load cases and only when Consented to by the Ministry. Post-earthquake serviceability, repair, and return to service criteria of Table 4.16 shall be demonstrated to the satisfaction of the Ministry. Installation of devices that can assist in the post-earthquake assessment of piles and in-ground plastic hinges are required.

Steel pipes subjected to in-ground hinging shall be made of steels satisfying the requirement of API Specification 5L, 46th Edition. Additional requirements for fabrication welds and splices are required for ASTM A252 material per the Special Provisions template.

***Commentary**:*

Table 4.4.5.4-1 is based on the AASHTO Guide Specifications for Seismic Bridge Design, 2011, by the American Association of State Highway and Transportation Officials, Washington, D.C., U.S.A. and this has been used with their permission.

For ERS and ERE requiring Consent of the Ministry, the designer should prepare a project specific seismic design criteria memo for the Ministry's review. The memo should cover items such as the rationale for selected ERS and ERE, energy dissipation mechanism, post-earthquake seismic behaviour (inspection, reparability, return to service), life cycle cost, constructability, durability, reliability, risks, maintenance requirements, any peer-reviewed experimental and/or analytical publications, relevant past projects, proposed seismic performance criteria, etc. The memo should be submitted as early as possible, e.g.: during concept development.

"Reasonably accessible" locations for in-ground hinges vary and should be considered on a case-by-case basis. For general guidance in typical situations, the Ministry considers "reasonably accessible" to mean less than 8 meters below the ground, mean water or tide level for steel piles and less than 4 meters for concrete piles below the ground, mean water or tide level.

If deep in-ground plastic hinges are expected, installation of devices such as inclinometer tubes in shafts can assist in the post earthquake assessment of the performance of the piles and pile hinges.

4.4.6 Performance-based design

4.4.6.1 General

Replace third paragraph with:

Lifeline bridges in Seismic Performance Category 2 and 3 require independent peer review unless otherwise specified by the Ministry.

The independent peer review shall be done by a firm other than the firm employing the Engineer of Record.

The independent peer review shall be done by recognised subject-matter experts acceptable to the Ministry in relevant fields, including but not limited to:

a) Performance-based structural seismic design of bridges, including foundations and supporting soils.
b) Performance-based geotechnical seismic design of bridges, including foundations and supporting soils.
c) Earthquake hazard definition and selection and modification of ground motions for use in nonlinear time history response analysis, including effects of soil-structure interaction.
d) Application of structural and geotechnical analysis software for use in nonlinear time history response analysis and interpretation of analysis results.

Commentary: *The requirement for independent peer review is in addition to Engineers and Geoscientists BC requirements for independent reviews, such as independent reviews of structural designs and independent reviews of high-risk professional activities or work. The scope of work for independent peer review is described in the Engineers and Geoscientists BC document entitled Performance-Based Seismic Design of Bridges in BC, Section 2.4 and in the Commentary to S6:19, Clause C4.4.6.1. Recognition for subject matter experts may be evidenced by one or more of the following accomplishments in the field of bridge earthquake engineering and seismic design: industry achievements substantially above those ordinarily achieved by practicing engineers; industry awards and/or acknowledgements pertaining to* *seismic engineering of bridges; active technical member invited to collaborate with a standards organisation or code committee pertaining to seismic engineering of bridges; leadership role in a centre of expertise dealing with earthquake-resistant design; peer reviewed publications addressing seismic engineering of bridges; and/or international solicitation as consultancy expert in the seismic engineering of bridges.*

4.4.6.3 Performance criteria

Replace the clause with:

The performance criteria for different performance levels are given in Table 4.16. The assessment of damage performance levels specified in Table 4.16 shall be carried out using nonlinear time history method or by using inelastic static pushover analysis up to the design displacement (see Clause 4.5.3.5). When assessing performance in performance-based design, the behaviour of the structural components may be determined using the expected material properties as defined in Clauses 4.7.2 and 4.8.2.2.

Table 4.16:

Add the following for “Immediate”, “Limited”, and “Service Disruption” Service:

Repair work shall restore the structure to meet the original design loading requirements.

Commentary: *The intent is the damage shall be repairable such that the repairs restore the structural capacity for its original design loads, both vertical and lateral loads.*

Effects of live load on bridge inertia mass shall not be included in the dynamic analysis.

Replace description for “Minimal Damage” to:

Minimal Damage

- Live Load: 50% of unfactored multi-lane normal traffic (without Dynamic Load Allowance) shall be applied concurrent with seismic demands.
- General: Bridge shall remain essentially elastic with minor damage that does not affect the service 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 the lesser of 0.003 and $1.5\varepsilon_{ye}$. Local or global buckling shall not occur.
- Connections: The service performance level of 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 service 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 shall occur.

- 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.
- Steel Piles: For steel piles and the casing of concrete infilled steel pipes, steel strains shall not exceed the ε_{ye}. For concrete infilled steel pipes, concrete strains shall not exceed 0.004.
- Concrete Piles: Piles shall remain essentially elastic and concrete strains in concrete piles shall not exceed 0.004.

Replace description for "Repairable Damage" to:

Repairable Damage

- Live Load: 100% of unfactored multi-lane normal traffic (without DLA) in a minimum of 50% of the lanes, but not less than one lane in each direction, shall be applied concurrent with seismic demands. The operational traffic lanes shall be shown on the plans.
- General: The bridge may experience inelastic behaviour, but primary members shall be repairable in place.
- Concrete Structures: reinforcing steel tensile strains shall not exceed 0.025.
- Steel Structures: Steel strains shall not exceed the larger of 0.008 and $2/3\varepsilon_{sh}$. 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.
- Displacements: Residual displacement, settlement, translation or rotation, of the structure or foundations, including retaining and wing walls, shall not compromise the service performance level.
- 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 mitigate span loss in aftershocks. Damage to restrainer supporting elements such as end diaphragms or substructure shall not require bridge closure to repair.
- Steel Piles: for steel piles and the casing of concrete infilled steel pipes, steel strains shall not exceed the lesser of 0.003 and $1.5\varepsilon_{ye}$. For concrete infilled steel pipes, concrete strains shall not exceed 40% of its ultimate confined strain limits.
- Concrete Piles: Reinforcement strains shall not exceed 0.01 and concrete strains in concrete piles shall not exceed 0.006.
- Ground deformations shall be mitigated such that permanent foundation offsets are small and repair objectives can be met. Foundation offsets shall be limited such that repairs can bring the structure back to the original operational capacity.

Replace description for “Extensive Damage” to:

Extensive Damage

- Live Load: a minimum of one lane of unfactored normal traffic (without DLA) shall be applied concurrent with seismic demands for one- and two-lane bridges. One lane of unfactored normal traffic (without DLA) in each direction, but not less than 30% of lanes, shall be applied concurrent with seismic demands for bridges with more than two lanes. The emergency traffic lanes shall be shown on the plans.
- 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 loads, 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: Steel strains shall not exceed the lesser of 0.06 and 0.5 ε_{ue}. 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 restricted emergency traffic. Bearings and joints may require replacement.
- Restrainers: Restraining systems may be damaged 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.
- Steel Piles: For steel piles and the casing of concrete infilled steel pipes, steel strains shall not exceed the larger of 0.008 and 2/3 ε_{sh}. For concrete infilled steel pipes, concrete strains shall not exceed 80% of its ultimate confined strain limits.
- Concrete Piles: Reinforcement strains shall not exceed 0.025. Concrete spalling is permitted.

Replace description for “Probable Replacement” to:

Probable Replacement

- Live Load: 30% of unfactored multi-lane normal traffic (without DLA), shall be applied concurrent with seismic demands.
- 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.
- Steel structures: Steel strains shall not exceed ε_{ue}.
- Extensive distortion of beams and column panels may occur.
- Members shall be capable of supporting loads, 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, collapse of approach walls within the bridge approach embankment interface zone, nor prevent safe evacuation of the bridge.
- Steel Piles: For steel piles and the casing of concrete infilled steel pipes, steel strains shall not exceed the lesser of 0.06 and 1/2 ε_{ue}. For concrete infilled steel pipes, concrete strains shall not exceed its ultimate confined strain limits.
- Concrete Piles: Reinforcement strains shall not exceed 0.05. Concrete spalling is permitted but the confined core concrete shall not exceed 80% of its ultimate confined strain limit.

4.4.6.4 Consideration of aftershock effects

Delete the clause.

Commentary:

In general, superstructures, ductile substructures, restrainers and foundations designed to S6:19 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 and is capable of sustaining multiple additional cycles of seismic loading.

Although there are no standardized methodologies for the aftershock assessment, the potential effects of aftershocks on the performance levels of Lifeline and Major-Route bridges in SPC 3 may be needed if required by the Ministry.

4.4.7 Force-based design

4.4.7.1 General

Add the following:

For regular bridges of slab, beam-girder, or box girder construction, with a structurally continuous reinforced concrete deck designed as a horizontal diaphragm between substructure elements, and where the superstructure is not integral with the substructure, a detailed analysis of earthquake effects on superstructure components shall not be required. Cross-frames and diaphragms, bearings, bracing connections and connections between the girders at the abutments and piers shall be analysed.

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

Delete the last paragraph and add the following:

The effects of vertical ground motion for FBD and PBD shall be accounted for by using the load factors on dead load specified in Table 3.3 or by using a dead load factor of 1.0 and vertical effects arising from a dynamic analysis including vertical accelerations explicitly, in combination with the orthogonal effects described above.

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. The bi-direction combination applies for verifying the performance criteria of Clause 4.4.6.3.

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 degrees 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

4.4.10.2 Seismic Performance Category 1

Add the following:

The substructure shall be designed for the forces determined in accordance with this clause, or for the seismic design forces obtained from EDA or ESA.

Commentary: *A continuous and reliable load path to transfer all seismic inertial loads from point of application to surrounding soils is essential for earthquake resistance systems.*

4.4.10.4 Seismic Performance Category 3

4.4.10.4.2.2 Seismic design forces for capacity-protected elements for force-based and performance-based design

Delete the last paragraph and replace it with:

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 forces of 1.25, 1.35, and 1.5 times the elastic forces for Other, Major-Route, and Lifeline importance category respectively unless required otherwise by project-specific criteria. The elastic forces need not be greater than forces obtained from capacity design principles using probable resistance. Connectors shall be designed to transmit, in their restrained directions, the maximum force effects determined from the elastic seismic forces using the above increase, but these forces need not exceed the force that can be developed by the ductile substructure element attaining 1.25 times its probable resistance.

Commentary: *The design of columns or pier walls which are part of the energy dissipating system are sometimes governed by other load cases rather than seismic load cases. As such, the probable moment resistance of these components may be greater than the elastic seismic demands. Consequently, capacity design based on the forces obtained from probable resistance tends to be overly conservative. To maintain enough margin of the safety between the brittle failure modes and ductile behaviour modes, the elastic seismic forces need to be increased. The margin of safety is provided as a function of the importance designation of the bridge.*

4.4.10.4.3 Yielding mechanisms and design forces in ductile substructures

Add the following to the fourth paragraph:

Design forces for ductile substructure elements for Major-Route and Lifeline importance categories shall be taken as the unreduced elastic design forces increased by 1.35 and 1.5 for respectively, unless required otherwise by project-specific criteria.

Commentary: *Note that "reasonably accessible" for inelastic hinge locations is described in Clause 4.4.5.4.*

4.4.10.7 Hold-down devices

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.*

4.5 Analysis

4.5.3 Multi-span bridges

4.5.3.5 Static pushover analysis

Add the following:

The static pushover analysis shall be taken to the expected displacement demands considering the bi-directional effects of seismic loading. Displacement demands shall 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.

Commentary:

Static pushover 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 pushover 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.

4.7 Concrete structures

4.7.4 Seismic Performance Category 2

Replace the second paragraph with the following:

The transverse reinforcement at the top and bottom of a column and in potential plastic hinge zones of beams, columns, shafts 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

4.7.5.2.4 Column shear and transverse reinforcement

Commentary:

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

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 presented in "Displacement-based Seismic Design of Structures, Priestley, Calvi and Kowalsky, IUSS Press, 2007".

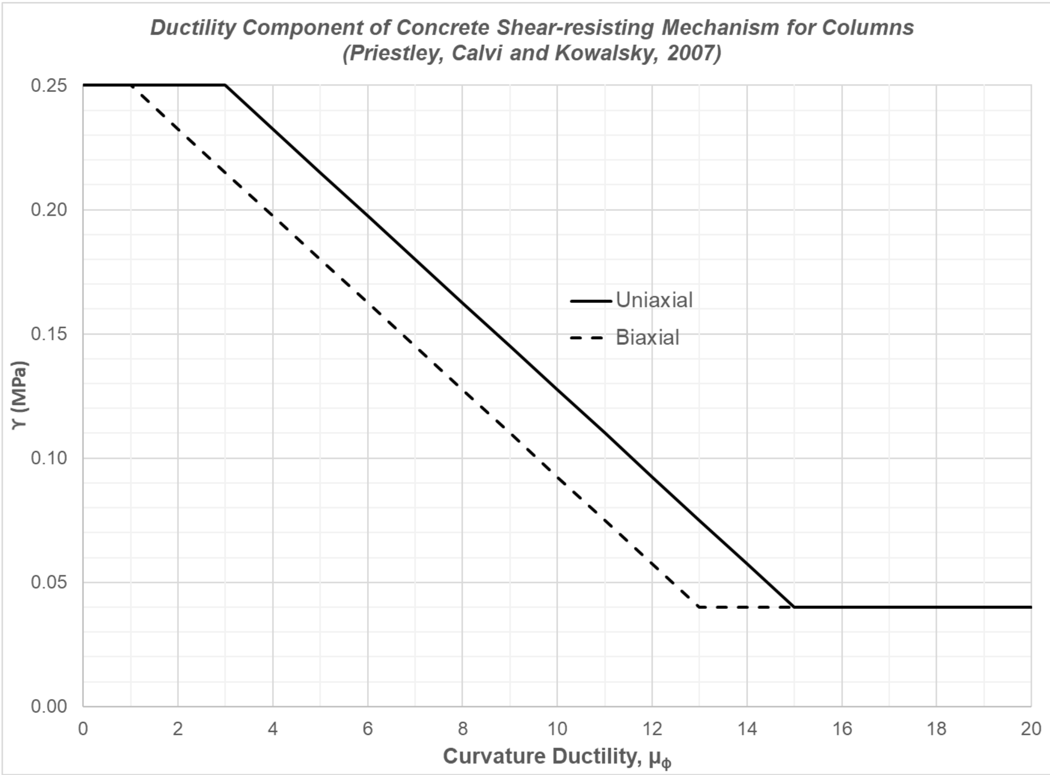
$$V_n = \phi_C V_C + \phi_S V_S + V_P$$

V_c = Concrete Shear-Resisting Mechanism

$$V_C = \alpha\beta\gamma\sqrt{f'_{ce}}\,0.8A_g$$

$\alpha = 1.0$

$\beta = 0.5 + 20\rho_l \leq 1.0$



V_S = Steel Shear-Resisting Mechanism (based on spirals or ties crossed by a crack with cot θ measured from vertical, using θ = 35° for design)

For Rectangular Columns:

$$V_S = \frac{A_v f_y (D - c - c_o) \cot\theta}{s}$$

For Circular Columns:

$$V_S = \frac{\pi}{2} \frac{A_v f_y (D - c - c_o) \cot\theta}{s}$$

V_P = Axial Load Component

$$V_P = 0.85 P \tan\zeta$$

Definitions:

γ = factor for degradation in V_c with increasing curvature ductility, MPa
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 Depth / 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
θ = 35° for design
P = axial load from bridge weight plus plastic mechanism effects
ζ = angle of inclination of a compression strut through the column, measured from the member's longitudinal axis

Plastic Hinge Region in Tall Columns or those having high axial loads:

"Tall" columns consider those with clear height to column diameter (H/d), or to least rectangular dimension, greater than 15.

"High axial loads" considers those with greater than 30% of the crush load (f'_cA_g) 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.

4.7.5.2.5 Transverse reinforcement for confinement at plastic hinge regions

4.7.5.2.7 Splices

Replace the first sentence of the second paragraph with:

Lap splices in longitudinal reinforcement and lap splices in spiral reinforcement shall not be located in plastic hinge regions. "No-Splice Zones" shall be clearly identified on the plans.

Add the following at the end of the third paragraph:

Welded splices and mechanical splices shall only be used when Consented to by the Ministry.

Commentary:

For long plastic hinge regions, the Ministry may accept mechanical splices in longitudinal reinforcements in plastic hinge regions based on the following:

- *If the longest length of commercially available reinforcement steel (not less than 18 m) is used to ensure that splicing is avoided where possible and minimized elsewhere, and*
- *In addition to the requirements of Section 8.4.4.4, the mechanical couplers shall be capable of developing 125% of the maximum tensile strength of the spliced bars.*

4.7.5.4 Column connections

Add the following after the first paragraphs:

For Lifeline and Major-Route bridges in Seismic Performance Category 3, the design of column connections, including member proportions, details, and reinforcement, shall be designed as capacity-protected elements based on beam-column joint design methodologies as described in Caltrans Seismic Design Criteria Version 2.0. Headed bars may only be used when Consented to by the Ministry.

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 Section 8 of S6:19.*

Caltrans provides additional guidance for the use of headed bars in Memo to Designers 20-21 "Seismic Requirements for Headed Bar Reinforcement", 2016.

4.8 Steel structures

4.8.3 Sway stability effects

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

4.8.4 Steel substructures

4.8.4.4 Seismic Performance Category 3

4.8.4.4.5 Buckling restrained braced frames

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

4.10 Seismic base isolation and supplemental damping

4.10.4 Performance based design

4.10.4.3 Performance criteria

Delete Table 4.19 and replace with:

Unless Consented to by the Ministry, the displacement capacity of isolator and damping units shall not be less than 1.5d plus offset displacement and shall be determined from prototype testing. Displacement capacity is defined as the displacement that can be achieved without failure. Failure includes but is not limited to the following:

- Component shear failure,
- Component bond failure,
- Surface cracks on elastomers wider or deeper than two-thirds of the cover thickness,
- Material peeling,
- Scoring of stainless steel plate,
- Permanent deformation, or
- Leakage.

Unless Consented to by the Ministry, no strain hardening shall occur prior to 1.25*d* plus offset displacement.

4.10.5 Analysis procedures

4.10.5.3 Uniform-load and single-mode spectral analysis

Replace item d) in first paragraph with:

d) the bridge is not located in a seismic site designation X_F.

4.10.6 Design displacements for seismic and other effects

Delete the second paragraph and replace with the following:

The offset displacement is the resultant of the displacements in each of the two orthogonal directions due to 50% of the deformations due to temperature changes and 100% of the deformations induced by concrete shrinkage and creep.

Commentary: *Deformations from creep and shrinkage may be determined considering the timing of bearing placement or re-setting to mitigate such effects if shown on the plans. The value of 100% for thermal demands in S6:19 in combination with seismic demands for 5% or 10% in 50 years is believed conservative and uncalibrated.*

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 S6-19 as modified in this Supplement.

Since S6-14, S6 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 S6:19 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 shall be evaluated based on performance-based principles using seismic performance levels and 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 and evaluation

Add the following paragraph:

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 baseline hazard for seismic evaluation of existing bridges prior to S6-14 code 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

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

4.11.5.1 General

Delete the second paragraph.

4.11.5.2 Limited evaluation

Delete clause.

Commentary: *The prescribed limited evaluation shall not be used for the seismic assessment or retrofit of Ministry bridges. 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 method using ISPA or NTHA 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 the following after the second paragraph:

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.*

4.11.9 Required response modification factor for force-based design approach

Delete clause.

4.11.10 Response modification factor for existing substructure elements

Delete clause.

4.11.12 Bridge access

Modify sentence by deleting “for bridges located in Seismic Performance Category 3”.

Add the following paragraph:

Damage to embankments and abutments shall be evaluated.

4.11.13 Liquefaction of foundation soils

Delete first paragraph, including sub-clauses (a) and (b), and replace with the following:

The potential for liquefaction of the foundation soils shall be evaluated as required to determine performance.

Add the following clauses:

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 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 specified by the Ministry.
- Description of the 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.
- A description of the 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 [typically an elastic Load Path 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 vulnerable components for the current structure, that could affect use, 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, 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

Add the following paragraphs at the beginning of the clause:

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

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.

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.*
- *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.*

4.12.2 Response modification factor for force-based design approach

Delete the clause.

Commentary: *The Ministry does not use force-based design for seismic rehabilitation.*

Add the following clause:

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 test hole/test pit 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.