SEISMIC RETROFIT DESIGN CRITERIA

British Columbia
Ministry of Transportation

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Prepared by:	Don Kennedy, P.Eng., Associated Engineering (BC) Sharlie Huffman, P. Eng. Bridge Seismic Engineer
Recommended by:	Kevin Baskin, P. Eng. Chief Bridge Engineer
Approved by:	Dirk Nyland, P. Eng. Chief Engineer

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1 GENERAL

1.1 INTRODUCTION

The Ministry of Transportation (MoT) has established a seismic risk reduction policy for all highway bridges in British Columbia. 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.

MoT has designed bridges to meet stringent earthquake design standards since 1983. These newer bridges may sustain damage, but are not expected to collapse in a major earthquake. Those structures designed or built prior to 1983 are considered potentially vulnerable to collapse or major damage from earthquakes.

In 1989, MoT 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" (See Section 1.3).

1.2 SCOPE AND APPLICATION

This document, "Seismic Retrofit Design Criteria" (SRDC), specifies requirements for seismic assessment and retrofit design of bridges in British Columbia. It emphasizes structural aspects of seismic performance, including the effects on structures arising from liquefaction and seismically induced ground deformations. This document does not address other potential risks, such as landslides or tsunamis.

Section 2 provides a summary of Ministry policies and strategies for seismic retrofitting. Sections 3 through 7 specify minimum seismic design criteria for the upgrading of Lifeline bridges, Disaster Response-Route bridges, bridges on Economic Sustainability Routes, and

Other bridges (definitions of these terms are provided in Section 2.1). Section 8 provides requirements and guidelines for a seismic retrofit strategy report.

For design criteria not specifically addressed in this document, assessment and retrofit design shall be in accordance with the references provided in Section 1.3, or in project-specific criteria specified by the Ministry.

Designers using this document on Ministry projects must be experienced in the seismic design, assessment, and retrofit of bridges, must exercise engineering judgment in the application of these criteria, and must be registered as a Professional Engineer in British Columbia.

1.3 DESIGN CODES AND STANDARDS

Unless specified otherwise, the following reference design codes and standards shall be used, amended as required herein. These standards are listed in their order of precedence:

a) BC Ministry of Transportation Supplement to CAN/CSA-S6-00, 2005.

b) CAN/CSA-S6-00, Canadian Highway Bridge Design Code, Canadian Standards Association, 2000.

c) Improved Seismic Design Criteria for California Bridges: Provisional Recommendations, Report No. ATC-32, Applied Technology Council, 1996.

The following guide documents should be used to supplement the above standards:

a) Seismic Design and Retrofitting of Bridges, Priestley, M.J.N., Seible, F., and Calvi, G.M., Wiley-Interscience, John Wiley and Sons Inc., 1996.

b) Design Guidelines for Assessment, Retrofit and Repair of Bridges for Seismic Performance, Priestley, M.J.N., Seible, F., and Chai. Report No. SSRP-92/01, University of San Diego.

c) CalTrans Memo to Designers 20-4, California Department of Transportation, Version 1.3 (Current updates May 14, 2004, www.dot.ca.gov/hq/esc/techpubs/manual/othermanual/other-engin-manual/seismic-design-criteria/sdc.html).

d) Seismic Retrofitting Manual for Highway Bridges, U.S. Department of Transportation, Federal Highway Administration (FHWA), May, 1995.

e) Bridge Seismic Retrofit Program, BC Ministry of Transportation & Highways, Engineering Branch, February 2000.

2 BRIDGE SEISMIC RETROFIT STRATEGY

Several retrofit options are available for protecting bridges against collapse and major damage from earthquakes. In general, the level of retrofit protection is selected based on the importance of the route and the structure, the site seismicity, and the required post-earthquake performance of the structure in terms of traffic access and acceptable damage.

The current retrofit policy comprises upgrading to a single-level design event (475-year return period event) using a staged upgrading of bridges on a priority basis. The first priority is to retrofit bridges in Seismic Performance Zone 4 and in higher risk areas of Seismic Performance Zone 3. Retrofit of other bridges, as well as a higher level of seismic retrofit of important bridges, will be undertaken as feasible.

The Ministry strategy for seismic retrofitting is outlined in the following sections. Bridge importance classifications are described in Section 2.1. The definition and application of retrofit levels are described in Section 2.2. Seismic Performance Criteria are specified in Section 2.3.

2.1 BRIDGE CLASSIFICATIONS

Bridges that are currently candidates for seismic retrofitting have been classified into four importance categories as follows:

- Lifeline Bridges.
- Disaster Response Route Bridges ("Emergency Route" bridges within S6-00).
- Economic Sustainability Route Bridges ("Other" bridges within S6-00).
- Other Bridges.

These classifications have been made on the basis of social/survival and economic recovery requirements. The bridge classification for each project is specified in the Terms of Reference. Bridge classifications in this Document are consistent with classifications defined in S6-00.

2.1.1 Lifeline Bridges

Lifeline bridges are unique or major structures, for example the Port Mann Bridge, Oak Street Bridge, and Second Narrows Bridges. These bridges are most critical to emergency response capability and post-earthquake economic recovery. The time to restore these structures to functional performance after closure would have a major economic impact.

These bridges are being upgraded to a minimum 'safety' level retrofit (refer to Section 2.2) during this stage.

2.1.2 Disaster Response Route Bridges

In the Lower Mainland and on Vancouver Island a system of routes have been designated as Disaster Response Routes (DRRs). Disaster Response Routes are corridors that must be kept open for emergency vehicle response following a major earthquake.

Lifeline bridges and bridges on Disaster Response Routes are being retrofitted as the highest priority in the first phase of the retrofit program. A detailed outline of the two phases of the retrofit program is provided in Reference 1.

2.1.3 Economic Sustainability Route Bridges

Systems of important highways have also been designated as Economic Sustainability Routes (ESRs) in the Lower Mainland and on Vancouver Island. These routes are considered essential to maintaining minimum effective transportation levels for economic purposes following a major earthquake. in general, bridges on these routes are being given the next highest priority for retrofitting and will be addressed in the second phase of the program.

ESR bridges are to be treated as “Other” bridges for the application of provisions within S6-00.

2.1.4 Other Bridges

Other Bridges are generally those bridges that are least critical to emergency response capability and post-earthquake economic recovery. A prioritization methodology has been developed and is being used to establish the retrofit ranking of these bridges. This classification is considered to be consistent with the corresponding level within S6-00.

2.2 APPLICATION OF SEISMIC RETROFIT LEVELS

Levels of seismic retrofitting shall be as specified in Table 2.2. The retrofitting levels specified in Table 2.2 are provided for guidance as it may be found that some bridges currently meet the Seismic Performance Criteria specified in Section 2.3 and therefore do not require retrofitting. Definitions of the retrofitting levels in Table 2.2 are provided below. The Seismic Performance Zone and level of retrofitting for each project are specified in S6-00 and the Terms of Reference.

Table 2.2
Application of Seismic Retrofitting Levels

Bridge Classification	Retrofit Level	
	Current Stage	**Possible Future Stage**
Lifeline Bridge		
Seismic Performance Zone 4	Safety 2	Functional
Seismic Performance Zone 3	Safety 2	Safety 2
Seismic Performance Zone 2	Safety 1	Safety 2
DRR Bridges		
Seismic Performance Zone 4	Safety 2	Safety 2
Seismic Performance Zone 3	Safety 1	Safety 2
Seismic Performance Zone 2	Superstructure	Safety 1
Seismic Performance Zone 1	None	None
ESR Bridges		
Seismic Performance Zone 4	Safety 1	Safety 2
Seismic Performance Zones 3, 2	Superstructure	Safety 1
Seismic Performance Zone 1	None	None
Other Bridges		
Seismic Performance Zones 4, 3, 2	Superstructure	Safety 1
Seismic Performance Zone 1	None	None

Superstructure retrofitting prevents superstructure collapse during the design earthquake resulting from the unseating of bridge superstructures by tying spans to supporting piers or abutments. Devices such as bearings (with appropriate restraint), cables or threadbars, shear keys, seat extensions, girder extensions, integral connections, or other methods may be considered.

This level of retrofit does not ensure "collapse prevention" in that the potential for failure of substructure components is not addressed. Superstructure retrofitting is expected to be much less expensive than Safety retrofitting and yet significantly reduce the risk of collapse. Therefore, it provides the greatest increase in overall safety for a given expenditure, and may be undertaken as a first phase of retrofit in a given bridge or inventory of bridges.

Safety retrofitting is defined as the prevention of collapse of all or part of the bridge during the design earthquake. Two levels of "Safety" retrofits are identified below.

Safety 1 (S1): This level of retrofit is a collapse prevention upgrade comprising a superstructure retrofit and prevention of serious structural deficiencies in substructures. Ground improvements are generally not expected to be performed for this level of retrofit.

Safety 2 (S2): This level of retrofit is a collapse prevention upgrade addressing all potential failure modes. Ground improvements, if necessary to meet the performance requirements, are also considered to be part of this level of retrofitting. Current bridge codes require ordinary new bridges ("other" bridges in S6-00) to withstand the same 475-year earthquake without collapse and with repairable damage. "Safety 2" retrofitting therefore targets a level of safety and performance roughly comparable to that of ordinary bridges designed to S6-00.

Functional retrofitting requires that important (Lifeline and DRR) bridges remain in service after the design earthquake. It is comparable to more rigorous performance objectives contained in S6-00 for important new bridges.

The Ministry is not anticipating functional retrofitting in the current stage of retrofitting; it may be considered in a future stage.

2.3 SEISMIC PERFORMANCE CRITERIA

Bridges shall be assessed and their retrofits designed to meet one of the seismic performance criteria specified in Table 2.3, expressed in terms of the service levels and damage levels defined below. The Design Earthquake motion is defined in Section 3. Performance is assessed against this single level event. The required retrofit level, Service Level, and Damage Level are specified in the Terms of Reference for each project.

Table 2.3
Seismic Performance Criteria

Retrofit Level (and Bridge Application)	Seismic Performance Criteria (for Design Earthquake)	
	Service Level	**Damage Level**
Functional	Immediate	Minimal
Safety 2 (S2)	Limited	Repairable
Safety 1 (S1)	Significantly limited	Significant (no collapse)
Superstructure	Possible loss of service	Significant (loss of span prevention)

A higher level of retrofitting may be specified in the current stage of retrofitting on a case-by-case basis. in cases where a significant improvement in earthquake performance can be achieved at an acceptable incremental cost, the retrofit level may be upgraded in the first stage of work to provide an improved seismic performance level.

2.3.1 Service Levels and Damage Levels

Service levels and damage levels are defined as follows:

(a) Service Levels

- **Immediate:** Full access to normal traffic is available within hours following the earthquake.

- **Limited:** Limited access (e.g. reduced or designated lanes, emergency traffic). It is recognized that approximately 24 hours may be needed to complete a post-earthquake inspection of the bridge. Full access to normal traffic is to be restorable within days.

- **Significantly Limited:** It is expected that limited access to emergency traffic is possible within days following the earthquake. Public access is not expected until repairs are completed.

- **Possible loss of service:** Access to traffic is not envisaged for a prolonged period.

(b) Damage Levels

- **Minimal Damage:** No risk of collapse. Essentially elastic performance. The following behaviour is intended: Minor inelastic response is limited to narrow cracking in concrete without concrete spalling, or minor yielding or local buckling of secondary steel elements. Permanent offsets associated with plastic hinging or non-linear foundation behaviour are not present.

- **Repairable Damage:** No risk of collapse. Damage that can be repaired without compromising the required service level. Inelastic response is expected but will be limited such that the structure can be restored to its pre-earthquake condition without replacement of primary structural members or requiring complete closure. Permanent offsets (residual displacements) are not to exceed approximately 0.5% or impede the required repairs.

  For concrete portions of the structures, inelastic response may result in concrete cracking, reinforcement yielding (not fracture or buckling), and minor spalling of cover concrete. for steel structures, inelastic response shall not result in member fracture or connection failure for primary load carrying members; however, limited buckling of secondary steel members, and limited flexural yielding in steel columns or limited axial tensile yielding of braces may occur in primary members.

Earthquake induced foundation movements or other foundation effects are acceptable if such effects can be repaired to restore the structure to full service, and if the repairs can be performed under the traffic service levels specified in Table 2.3.

- **Significant Damage (No Collapse):** Damage that would require closure to repair is expected.

  Damage does not cause collapse of any span or part of the structure, nor lead to the loss of the ability of primary support members to sustain gravity loads. Permanent offsets may occur and damage consisting of cracking, yielding, and major spalling of concrete may require closure. Re-instatement of the structure may require extensive repairs and potentially the re-construction of bridge components. For concrete structures, inelastic response may result in significant cracking, yielding of reinforcing bars, and major spalling of concrete.

  For steel structures, inelastic response shall not result in fracture or connection failure for primary load carrying members; however, buckling of secondary steel members and significant flexural yielding of steel columns or significant axial tensile yielding of braces may occur in primary members.

  The following foundation behaviour is intended: Earthquake induced foundation movements or other earthquake induced foundation effects shall be considered acceptable if the performance of the structure satisfies the seismic performance requirements specified in Table 2.3. Expected inelastic deformations of the foundations shall be determined and the effects of these movements on the performance of the structure shall be considered in the evaluation and retrofit design.

- **Significant Damage (Loss-of-span Prevention):** A risk of extensive, non-repairable damage is accepted, but collapse of the superstructure from unseating or from structural failure of supporting piers is not acceptable. The potential for failure from ground deformations is recognized but not necessarily addressed.

3 SEISMIC LOADING

3.1 DESIGN EARTHQUAKES

a) A single level Design Earthquake shall be used for structural retrofits. The Design Earthquake is represented as a probabilistically assessed ‘uniform hazard’ response spectrum (UHRS) with a 10% probability of exceedence in 50 years. This corresponds to a ground motion with a 475-year return period. The firm ground and modified (soil) design spectra, and the requirements for time history records representing this Design Earthquake are described below.

The duration of strong shaking and the total duration of the selected records shall be consistent with earthquakes of approximate magnitude 6.5 to 7.0 for the 475-year return period event.

b) In addition, a Cascadia subduction event shall be considered for the assessment of liquefaction and its effects on the structure. The duration and intensity of shaking shall be consistent with a magnitude in excess of 8.0.

3.2 DESIGN RESPONSE SPECTRA, GROUND MOTION TIME HISTORIES AND EARTHQUAKE MAGNITUDE

Table 3.1 defines the minimum ground motion requirements for bridges within the various classifications noted above.

Table 3.1
Minimum Ground Motions

Minimum Ground Motion Requirements				
	Firm Ground Spectra	**Soil Spectra**	**Time History Records**	**Vertical Seismic Inputs**
Lifeline Bridges	UHRS [1]	Dynamic site Response	3 sets, spectrum-compatible [5]	2/3 of firm ground [6]
DRR Bridges	UHRS[1] or	Dynamic Site Response [3] or	3 sets, spectrum-compatible	Not required
	S6-00 [2] with I-1.5	S6-00[2,4] with I = 1.5	Not required	
ESR and Other Bridges	UHRS[1] or	Dynamic site response [3] or	Not required	Not required
	S6-00 [2] with I-1.0	S6-00 [2,4] with I = 1.0		

Notes to Table 3.1:

1. UHRS as described in Section 3.2.1 (a).
2. Design spectrum as defined in CAN/CSA S6-00 Clause 4.4.7.1
3. Site response analysis as described in Section 3.2.1 (b).
4. Soil factor “S” as specified in S6-00 and as appropriate for the site. Geotechnical engineering input is required unless authorized otherwise by the Ministry.
5. Time history records as described in Section 3.2.2.
6. Vertical ground motion allowance as described in Section 4.3.5. Vertical seismic inputs, where required, are applicable to time history and response spectrum analysis, and in lieu of more detailed information may be taken as 2/3 of the firm ground spectra or firm ground horizontal motions.

3.2.1 Design Response Spectra

For analysis and design using modal spectral dynamic analysis, the design earthquake shall be characterized by five-percent-damped design response spectra. The design response spectra shall be as follows:

(a) Firm Ground Design Response Spectrum

The firm ground design response spectrum for horizontal loading shall be the site-specific Uniform Hazard Response Spectrum (UHRS) for the 10% in 50 year probability of exceedence (50th percentile (median) data) obtained from the Geological Survey of Canada (GSC UHRS - Pacific Geoscience Centre). The GSC model shall be that used as the basis of the seismicity developed for the 2005 National Building Code of Canada.

The GSC UHRS provides spectral values for periods of response up to 2 seconds. If required, spectral acceleration values for higher periods shall be extrapolated by assuming the spectral acceleration reduces according to a 1/T relationship, or as otherwise approved by the Ministry.

The GSC uniform hazard response spectrum exceeds the response spectrum obtained from an offshore d earthquake (up to a period of about 4 seconds) for most locations in B.C. Therefore, a subduction earthquake event shall be considered mainly for the assessment of liquefaction and related effects unless otherwise specified by the Ministry in project-specific criteria.

(b) Modified Design Response Spectrum

Modified surface design response spectra shall be used as required where layers of softer soils overlie firm ground.

Modified design response spectra for horizontal loading shall be developed using dynamic site response analysis. The firm ground design response spectra in 3.2.1 (a) and the spectrum-compatible ground motion time histories for firm ground obtained in 3.2.2 (a) shall be used in the dynamic site response analysis. Design spectral values shall be taken as the maximum values calculated from the three sets of input ground motion time histories.

3.2.2 Ground Motion Time Histories

When required for analysis and design using either elastic or inelastic time-history dynamic analysis, or for dynamic site response analysis, the design earthquake shall be characterized by design ground motion time histories as follows:

a) Firm Ground Records

A minimum of three sets of two orthogonal, horizontal ground motion acceleration time histories shall be developed for firm ground conditions, and shall be compatible with the firm ground design response spectrum specified in 3.2.1 (a).

Each set of ground motions shall be developed from two recorded orthogonal, horizontal earthquake records recorded at the same site from the same earthquake event. These records shall be modified so that their response spectra match the firm ground design response spectrum specified in 3.2.1 (a).

(b) Modified Ground Motion Time Histories

Modified design ground motion acceleration time histories at the foundation levels shall be developed as required where firm ground is overlain by layers of softer soils. The “foundation level” may be taken as the footing or pile cap level, at the ground surface level, or as specified or accepted otherwise by the Ministry. A more refined approach, such as variable inputs along the length of piles, may also be adopted.

A minimum of three sets of two orthogonal, horizontal ground motion acceleration or displacement time histories shall be developed using the firm ground motions described in Section 3.2.2(a).

The ground motion time histories shall be developed from two recorded orthogonal, horizontal earthquake records recorded at the same site from a past earthquake event. These orthogonal records shall be modified so that their response spectra match the surface design response spectrum specified in 3.2.1 (b).

3.2.3 Load Factors and Load Combinations

The load factors and load combinations for seismic design shall be as follows:

1.0D + 1.0EQ

where: D = Dead Load
EQ = earthquake loading specified in Section 3.

Combinations of earthquake effects in orthogonal directions shall be as described in Section 4.3.5.

4 SEISMIC ANALYSIS AND DEMANDS

4.1 GENERAL

The following section specifies minimum requirements for seismic assessment and retrofit design. For design criteria not specifically addressed in this section, design shall be in accordance with criteria developed for each bridge on a case-by-case basis as outlined in Section 1.3.

In cases of inconsistency between the requirements provided in this Section and the intent of the seismic performance criteria specified in Section 2.3, the performance criteria specified in Section 2.3 shall govern.

4.2 STRUCTURAL BEHAVIOUR

For analysis and design purposes, the structure shall be categorized according to its intended structural behaviour under horizontal seismic loading. Categories are defined in (a) through (d) as follows.

(a) Full-Ductility or Limited-Ductility Structure

Under horizontal loading from the Design Earthquake a plastic mechanism is expected to develop. The plastic mechanisms shall be defined clearly as part of the retrofit design. The retrofit design shall be such that expected yielding in substructure elements, other than piles, is restricted to locations that are accessible for inspection following an earthquake. Inelastic or other non-linear action should be limited to:

- Flexural plastic hinges in columns or pier walls.
- Yielding of braces in braced steel frame substructures.
- Inelastic soil deformation behind abutment walls and wingwalls.

Yielding of piles or inelastic soil behaviour adjacent to piles may be considered where yielding in the above components is not practical or economic, and if approved prior to retrofit design by the Ministry.

Details and proportions of as-built and retrofit components shall ensure appropriate deformation capacity under load reversals without significant strength loss. This shall be demonstrated by appropriate analyses and calculations.

(b) Elastic Structure

This is a structure that is intended to remain essentially elastic up to the design load from the Design Earthquake. A margin of reserve strength prior to the formation of brittle or other unacceptable failure modes is required.

(c) Structure with Protective Systems

This is a structure incorporating seismic isolation, passive energy dissipating devices, or other mechanical devices to control seismic response. Inelastic deformation is mainly concentrated in the energy dissipating devices.

(d) Structure with Rocking Response

This is a structure allowed to have a rocking response during the design earthquake. Rocking response of the structure shall be stable under dead and seismic loads, and the effects of impacts at the supports during rocking response shall be accounted for in the design.

The rocking response approach requires approval by the Ministry prior to implementation.

4.3 ANALYSIS

4.3.1 Minimum Analysis Requirements

The structure shall be analyzed and designed to satisfy equilibrium and kinematics using probable material properties and effective component stiffnesses. Dynamic analyses shall emphasize seismic deformations, and modelling and component properties shall ensure that deformation demands are realistically identified. Analyses shall demonstrate that appropriate strength and deformation reserves are provided such that the structural behaviours of Section 4.2 are achieved and that the seismic performance criteria of Section 2.3 are satisfied.

The minimum analysis requirements for the structure shall be as specified in Table 4.3.1.

Table 4.3.1
Minimum Analysis Requirements

STRUCTURAL BEHAVIOUR	REQUIRED ANALYSES
Full ductility, limited ductility or elastic	• Elastic dynamic analysis • Non-linear static analysis or non-linear dynamic analysis*
Structure using protective systems	• Elastic dynamic analysis satisfying 4.3.1.3(a) and non-linear dynamic analysis
Structure with rocking response	• Elastic dynamic analysis using substitute structure or non-linear dynamic analysis*

* If the structure is designed to remain elastic for the design earthquake only elastic dynamic analysis is required, with demands scaled up as noted in Section 4.5.2.

In all cases the expected horizontal, vertical, and rotational movements of the foundations, including those arising from non-linear foundation behaviour, shall be determined and the effects of these movements on the performance of the structure shall be considered in the analysis and assessment. Except for structures intended to remain elastic, modelling and analyses shall emphasize deformation demands in existing sub-structures.

Methods of analysis are described in this section and limits on design displacements are provided in Section 4.4.

4.3.1.1 Full-Ductility or Limited-Ductility Structure

(a) The initial design may be based on an elastic dynamic analysis (4.3.2). Design displacements shall be in accordance with 4.4.1.

(b) Non-linear dynamic analysis (4.3.4) may be used to refine design requirements determined in 4.4.1.1(a) subject to limits in Section 4.4.1.2 and 4.5.

(c) Non-linear static analysis (4.3.3) or non-linear dynamic analysis (4.3.4) shall be used to verify the minimum global displacement capacity of frames and the overall structure in the evaluation of the bridge.

4.3.1.2 Elastic Structure

(a) The design may be based on an elastic dynamic analysis (4.3.2). Design demands shall be in accordance with 4.4.2 and 4.5.2.

4.3.1.3 Structure with Protective Systems

(a) The initial design may be based on equivalent linear modal spectral analysis using increased effective period and equivalent viscous damping for the isolated modes.

(b) Non-linear dynamic analysis (4.3.4) modelling of the non-linear deformation characteristics of the protective systems shall be used to verify the design in accordance with Table 4.3.1.

(c) Design displacements shall be in accordance with 4.3.3 and shall be compatible with isolator requirements.

4.3.1.4 Structure with Rocking Response

The evaluation of design forces and displacements shall take into account the non-linear behaviour of the rocking response. Linear dynamic analyses, when used, shall use a substitute structure approach in which a secant stiffness is substituted to model the rocking response at the peak displacements. Damping shall be taken as not more than 5% of critical unless energy dissipating mechanisms are available.

4.3.2 Elastic Dynamic Analysis

4.3.2.1 General

As a minimum, elastic dynamic analysis shall be a modal spectral analysis. Modelling and analyses shall emphasize the determination of displacements. Foundation flexibility, bearings, and other sources of flexibility shall be considered.

Elastic time history analysis may be used to refine the design requirements, subject to the displacement limits in Section 4.4.1.

The use of a substitute structure approach in which substructure and other lateral supporting component stiffnesses, effective periods and effective damping may also be used in lieu of the initial stiffness approach as outlined in ATC-32.

4.3.2.2 Analysis Model

Modelling shall be based on the approaches described in ATC-32 Clause 3.2.1.6.

The structural model shall include the effects of cracking on stiffness of reinforced concrete members and shall include soil-structure interaction. A probable range of soil stiffnesses shall be used to investigate the sensitivity of response to soil properties.

4.3.2.3 Modal Spectral Analysis

The elastic design response spectrum used in elastic modal spectral analysis shall be in accordance with 3.2.

The number of modes considered in the analysis shall be sufficient to include all critical response modes. The modal responses shall be combined using the Complete Quadratic Combination (CQC) method.

Responses in multiple directions shall be determined in accordance with 4.3.5.

4.3.2.4 Elastic Time History Analysis

The input ground motions used in the elastic time-history analysis shall be in accordance with 3.2. Design actions shall be taken as the maximum values calculated from the three sets of time-history ground motions.

Damping not more than five percent of critical shall be assumed for all critical modes. Higher damping values require justification by experimental evidence and analysis and approval from the Ministry. Time increments used in the analysis shall be sufficiently small to capture all the critical response modes. The appropriateness of the time step increment used shall be confirmed.

Responses in multiple directions shall be determined in accordance with 4.3.5.

4.3.3 Non-linear Static Analysis

4.3.3.1 General

Non-linear static (pushover) analyses are used to determine the deformation capacity and patterns of inelastic behaviour in frames (piers). Cracked section properties of all pier members shall be used unless demands are shown to be low (e.g., less than approximately 50% of nominal capacity) over the entire lengths of members. Foundation flexibility and capacity shall be accounted for in the pushover analyses and in the determination of member ductilities.

4.3.3.2 Structural Model

The structural model shall be generally as described in Section 4.3.2.2. Plastic hinges, when predicted, shall be assumed to form at the ends of components adjacent to framing members or adjacent to local increases in member capacities.

The inelastic response characteristics of structural elements shall be justified by experimental evidence for structural elements with similar details or by stress-deformation or moment-curvature analysis. The behaviour of deficient structural details and deterioration shall be accounted for in the model.

4.3.3.3 Analysis Procedure

A step-by-step lateral displacement response (“pushover”) analysis of the structural model shall be performed. Seismic loads may be assumed to act in one horizontal direction only. The effects of gravity loads acting through lateral displacements shall be included where significant. Guidelines are provided in ATC-32, Clause 3.21.7.

4.3.3.4 Distribution of Loading

The centre of mass of the superstructure shall be displaced in steps to the minimum design displacement capacity specified in 4.4.1.2. Local displacements, hinge rotations or curvatures, and other relevant member demands shall be assessed at each step. The effects of inelastic strains on component capacities shall be included.

4.3.4 Non-linear Dynamic Analysis

4.3.4.1 General

Seismic response shall be determined using dynamic analysis techniques that consider non-linear stiffness and damping of the structure and soils. A probable range of soil stiffnesses shall be used to investigate the sensitivity of response to soil properties.

4.3.4.2 Structural Model

Minimum modelling requirements shall be as described in Section 4.3.2.2.
Non-linear dynamic analysis of structures with protective systems (seismic isolation and/or energy dissipating devices) shall use the non-linear force - deformation characteristics of the systems determined and verified by tests.

4.3.4.3 Analysis Procedure

Damping not more than five percent of critical shall be assumed for all critical modes except that higher total damping values may be acceptable where demonstrated, for example by hysteretic damping, in the time-history analyses. Degradation of component stiffness and damping under cyclic loading shall be included.

The sensitivity of the numerical solution to the size of the time increment used for the analysis shall be determined. A sensitivity study shall also be carried out to investigate the effects of variations in assumed material hysteretic properties.

Non-linear effects of gravity loads acting through lateral displacements shall be included where their effects are significant.

The model shall be analyzed for the input ground motions of the design earthquake of 3.2. Design actions shall be taken as the maximum values calculated for the three sets of input ground motions.

Responses in multiple directions shall be determined in accordance with 4.3.5.

4.3.5 Combination of Effects

4.3.5.1 General

Earthquake effects for Lifeline Bridges shall be determined using seismic input in three orthogonal directions. The orthogonal horizontal directions shall be the longitudinal and transverse axes of the bridge.

For Disaster Route bridges, Economic Sustainability Route bridges, and Other bridges, earthquake effects shall be determined using seismic input in the longitudinal and transverse axes of the bridge.

4.3.5.2 Procedures for Lifeline Bridges

Earthquake effects shall be combined as follows:

(a) For structures designed using modal analysis, seismic effects shall be determined for the following three load cases:

Seismic Load Case 1: Combine the longitudinal effects resulting from the longitudinal loading with 30 percent of the longitudinal effects arising from the transverse loading analysis and 30 percent of the longitudinal effects arising from the vertical loading.

Seismic Load Case 2: Combine the transverse effects resulting from the transverse loading with 30 percent of the transverse effects arising from the longitudinal loading and 30 percent of the transverse effects arising from the vertical loading.

Seismic Load Case 3: Combine the vertical effects resulting from the vertical loading with 30 percent of the vertical effects arising from the longitudinal loading and 30 percent of the vertical effects arising from the transverse loading.

(b) For time-history analysis the three orthogonal components of each set of ground motion time histories shall be applied simultaneously unless the structure length, changing foundation conditions, or other conditions indicate that multiple support analyses are warranted.

4.3.5.3 Procedures for Disaster-Route Bridges, Economic Sustainability-Route Bridges and Other Bridges

Earthquake effects shall be combined as follows:

(a) For structures designed using modal spectral dynamic analysis, seismic effects shall be determined for the following two load cases:

Seismic Load Case 1: Combine the longitudinal effects resulting from the longitudinal loading with 30 percent of the longitudinal effects arising from the transverse loading.

Seismic Load Case 2: Combine the transverse effects resulting from the transverse loading with 30 percent of the transverse effects arising from the longitudinal loading.

(b) For time-history analysis, the two orthogonal components of each set of ground motion time histories shall be applied simultaneously.

4.4 DESIGN DISPLACEMENTS

4.4.1 Full-Ductility or Limited-Ductility Structures

4.4.1.1 Global Displacement Demand

The global structure displacement is the total displacement at the effective centre of mass of the overall structure, subsystem, or frame at a particular location. It includes foundation displacements or rotations and elastic/inelastic deformations of individual members such as columns and cap-beams.

The global design displacement demand for full-ductility or limited-ductility structures shall be determined from either of the following two approaches:

(a) Displacements calculated from elastic dynamic analysis, or

(b) Displacements calculated from non-linear dynamic analysis. Design displacements obtained from the non-linear dynamic analysis shall not be less than 80 percent of the values obtained from the elastic modal spectral analysis of 4.4.1.1(a) unless it can be justified to the satisfaction of the Ministry that a lower response is acceptable. Design displacements obtained from non-linear dynamic analysis shall be used if they are greater than those obtained from elastic dynamic analysis.

4.4.1.2 Minimum Global Displacement Capacity

Either Non-linear Static Analysis (4.3.3) or Non-linear Dynamic Analysis (4.3.4) shall be used to verify the displacement capacity of frames and the overall structure in the evaluation of the bridge (see 6.2).

For retrofit design, the minimum global displacement capacity provided shall be at least 1.2 times the displacement obtained from 4.4.1.1 in the direction considered.

In addition to the minimum global displacement capacity outlined above, local ductility demands on ductile substructure elements shall not exceed element ductility capacities to ensure that the structure will meet the seismic performance criteria specified in Section 2.3.

4.4.2 Elastic Structure

Forces and displacements calculated from Elastic Dynamic Analysis shall be used in design. Forces shall be determined as specified in Section 4.5.2.

4.4.3 Structure with Protective Systems

Displacements calculated from Non-linear Dynamic Analysis shall be used in design. Foundation flexibility and non-linear response, including within foundations, shall be accounted for in the analyses. An appropriate range of soil properties shall be used in the soil-structure analyses.

4.4.4 Structure with Rocking Response

Displacement demands shall be taken as those from the analyses described in Section 4.3.1.4.

4.4.5 Design Displacements for Expansion Bearings

Unless longitudinal restrainers are provided as part of the design, displacement demands at expansion bearings shall be those determined from 4.4.1.1, 4.4.2, 4.4.3, or 4.4.4 multiplied by 1.5. In no case shall support lengths at expansion bearings be less than those specified in CAN/CSA-S6-00.

4.5 FORCE DEMANDS

The following requirements shall apply, depending on the intended structural action, as defined in Section 4.2.

4.5.1 Full-Ductility or Limited-Ductility Structures

The structure shall be evaluated or designed, using capacity design principles, for the internal forces generated when the structure reaches the required global displacement capacity or the forces generated when the structure reaches the intended plastic collapse mechanism. Forces from the non-linear static analysis or the design plastic mechanism shall be used, based on nominal material properties and with allowances for member overstrength as defined in this document.

Design forces for connections between the superstructure and bents, piers, and abutments, such as restrained directions of bearings and shear keys, may be determined as the lesser of 1.25 times elastic design forces or capacity design procedures (Section 6.2).

4.5.2 Elastic Structures

Design forces may be taken as the lesser of 1.25 times those obtained from elastic dynamic analysis (4.3.2) or obtained using capacity design procedures.

4.5.3 Structures with Protective Systems

Design forces shall be at least equal to those obtained from a non-linear dynamic analysis (4.3.4) except for connections of the protective systems to the superstructure and substructure. Design forces for these connections shall be at least 1.25 times those obtained from non-linear dynamic analysis unless approved otherwise by the Ministry.

5 CAPACITIES OF STRUCTURAL ELEMENTS (MEMBERS)

5.1 MEMBER CAPACITIES

Member capacities to be used for resisting seismic demands shall be the nominal resistance calculated with all material resistance factors taken as 1.0, except as modified in 5.2, 5.3, and 5.4.

5.2 MATERIAL STRENGTHS

The capacity of existing members for resisting seismic demands shall be based on the most probable (50th percentile) material strengths. Unless authorized otherwise by the Ministry, existing concrete and reinforcing steel strengths shall be determined using destructive testing methods, such as concrete coring and steel coupon testing.

5.3 NOMINAL RESISTANCE OF EXISTING CONCRETE ELEMENTS

The Nominal Resistance of existing concrete elements shall generally be evaluated in accordance with the requirements of CAN/CSA-S6-00 except that all material resistance factors shall be taken as 1.0. Component capacities may be determined using methodologies contained in the References cited in this Document.

For existing concrete members not satisfying all the design and detailing requirements of CAN/CSA-S6-00, account shall be taken of the effects of any such differences. Differences to be accounted for shall include, but are not limited to, the following:

a) Lightly reinforced existing concrete elements with reinforcing ratios lower than those required by CAN/CSA-S6-00.

b) Inadequately anchored or spliced reinforcing steel bars (flexural, shear or confining reinforcing bars).

c) The reduction in shear or joint shear capacity with increasing flexural ductility demand at plastic hinge locations.

5.3.1 Effects of Deterioration

The resistance of existing concrete elements shall be reduced to account for deterioration.

5.3.2 Ductility Capacity

The local ductility capacity (flexural rotational or curvature ductility) of concrete elements shall be determined at plastic hinge locations and shall take into account the following parameters:

1) Strain compatibility and moment-curvature analysis considering confinement and ultimate strains in reinforcing steel, cover concrete and core concrete. for existing reinforced concrete members with widely spaced ties (approximately 300 mm or greater) maximum strains are likely to be controlled by behaviour at the extreme fibre rather than at depth.

2) Reinforcing details (e.g., confinement reinforcing, anchorage or splice details).

3) Element type - primary or secondary element (member) - to account for the role of the element in the gravity load path during and after the design earthquake.

It will be necessary to calculate local ductility capacities for both existing and “as-retrofit” details, unless approved otherwise by the Ministry.

5.4 NOMINAL RESISTANCE OF EXISTING STRUCTURAL STEEL ELEMENTS

The Nominal Resistance of existing steel elements shall be evaluated in accordance with the requirements of CAN/CSA-S6-00 except that all material resistance factors shall be taken as 1.0.

For existing steel elements not satisfying all the design and detailing requirements of CAN/CSA-S6-00, account shall be taken of the effects of any such differences. Differences to be accounted for shall include, but are not limited to, the following:

a) For steel members with width/thickness (b/t) ratios exceeding those allowed by CAN/CSA-S6-00, the effect of local buckling shall be considered in evaluating their capacities.

b) Steel members whose slenderness ratios exceed those allowed by CAN/CSA-S6-00 shall be considered to act in tension only unless their behaviour under compression is evaluated based on verified research results.

For existing laced members, the effects of shear deformations on overall buckling strength shall be accounted for. for existing laced members, the effects of individual component buckling between laced points shall be accounted for.

5.4.1 Effects of Deterioration

The resistance of existing steel elements shall be reduced to account for any defects or deterioration.

5.4.2 Ductility Capacity

The local ductility capacity (flexural rotational capacity or curvature ductility capacity) of steel elements shall be determined and shall take into account the following parameters:

1) Plane section analysis considering material and ultimate strain properties;

2) Section type and properties (e.g., stiffened, compact, or non-compact sections);

3) Slenderness of the element; and,

4) Element type - primary or secondary element (member) - to account for the role of the element in the gravity load path during and after the design earthquake.

It will be necessary to calculate local ductility capacities for both existing and “as-retrofit” details unless approved otherwise by the Ministry.

6 RETROFIT DESIGN

6.1 GENERAL

The design of retrofits for members shall follow the design methodologies and guidelines outlined in this Document. The design of new members as part of retrofit works shall be to CAN/CSA-S6-00.

Design forces shall be as outlined below. Principles of capacity design shall be adopted where applicable.

6.2 CAPACITY DESIGN

Capacity design shall be applied to full-ductility or limited-ductility structures.

Locations of inelastic action shall be clearly identified in the retrofit design. The intended yielding mechanism shall be designed to form prior to any other failure mode due to overstress or instability in the structure and/or in the foundation (except for structures designed with a rocking response). Undesirable failure modes, such as shear failures in concrete columns and buckling of primary steel members such as columns or braces in a primary lateral load path, shall be avoided.

The structure shall be analyzed for the lateral forces that produce the intended plastic mechanism of the structure (the forces generated when the structure reaches the minimum global displacement capacity assessed as in Section 4.4.1.2 or the forces generated when the structure reaches the intended plastic collapse mechanism).

6.2.1 Overstrength Forces for Capacity Protected Elements

Forces on capacity protected elements shall be determined using overstrength demands associated with the design plastic mechanism. Capacity protected elements shall be designed to remain essentially elastic under these demands.

For the plastic hinging mechanism in columns or piers, minimum overstrength demands associated with plastic hinging shall be taken as:

- 1.3 times the nominal flexural resistance (minimum overstrength factor of 1.3) for concrete sections.

- 1.25 times the nominal flexural resistance (minimum overstrength factor of 1.25) for structural steel sections.

Higher overstrength factors shall be adopted where necessary to account for the effects of confinement, strain hardening at high strains, or other effects.

The overstrength moments, associated axial loads and shear forces, and moment distribution characteristics of the structural system shall determine the demands on capacity protected elements.

6.3 P-DELTA EFFECTS

Effects of gravity loads acting through lateral design displacements shall be included in the retrofit design.

6.4 DESIGN OF RESTRAINERS

Longitudinal restrainers include bars, ties, cables or other devices specifically designed for the purpose of limiting displacements at expansion bearings. Restrainer elements shall be designed to ensure integrity and ductility under excessive forces or movements without experiencing brittle failures. Friction shall not be considered as an effective longitudinal restrainer. The use of restrainers as retrofit measures should be considered in cases where sufficient seat lengths can not be economically provided to prevent loss-of-span failures arising from inertial loads or from ground deformation demands.

Restrainers shall be designed for forces as described in ATC-32, in Caltrans Seismic Design Criteria and relevant Memo to Designers, non-linear time history analyses, or as otherwise agreed by the Ministry. Forces associated with plastic hinging in ductile substructures may not provide sufficient restrainer capacity.

Detailing shall be such that thermal or other serviceability deformations are accommodated.

Restrainer connections and supporting elements shall be designed to resist not less than 1.25 times the ultimate restrainer capacity.

6.5 DECK SYSTEMS

The designer shall demonstrate that a clear, straightforward seismic load path to the substructure exists and that all components and connections are capable of resisting the imposed load effects consistent with the chosen load path. Minor permanent offsets are acceptable where unacceptable failure modes would not occur and where the performance objectives can be shown to be satisfied.

Diaphragms, cross frames, lateral bracing, and their connections that are identified as part of the load path transferring seismic forces from the superstructure to the bearings, and which are required to be designed as capacity-protected elements, shall be designed and detailed to remain elastic under the design earthquake, regardless of the type of bearings used. Slip in non-composite concrete deck to girder connections may be acceptable provided the performance objectives for the design earthquake can be shown to be satisfied.

The above requirements are not intended to preclude the use of yielding or energy dissipating end diaphragms as part of the retrofit design.

6.6 JOINTS AND BEARINGS

Bearing design shall be consistent with the intended seismic response of the whole bridge system and shall be related to the strength and stiffness characteristics of both the superstructure and the substructure.

Expansion bearings and their supports shall be designed and detailed to accommodate, in the unrestrained direction, the seismic displacements. Joint gland replacement following the design earthquake is acceptable.

Rigid-type bearings and their components shall be designed to remain elastic as capacity-protected elements during the design earthquake. for deformable-type bearings not designed explicitly as base isolators or fuses, selected ductile components may be allowed to yield during the design earthquake. Permanent offsets resulting from such yielding shall be accounted for in the design.

The design and detailing of bearing components resisting earthquake loads shall provide adequate strength and ductility. Guides or restraints in bearing systems shall either be designed to resist all imposed loads or an alternative load path shall be provided.

The friction resistance of bearing sliding surfaces shall be conservatively estimated (i.e., underestimated) where it contributes to resisting seismic loads, and shall be overestimated where friction results in the application of force effects to structural components as a result of seismic movements. A seismic load path comprising mainly bearing friction shall not be adopted unless authorized otherwise by the Ministry. Where friction is accepted, the potential for permanent offsets shall be considered and the performance requirements of Section 2.3 shall be demonstrated.

7 FOUNDATIONS

7.1 SEISMIC DESIGN PHILOSOPHY

The geotechnical design shall be consistent with the seismic performance criteria given in Section 2.3. These criteria include permanent vertical, horizontal and rotational foundation movements. Approach embankments shall be considered where they form a necessary access to the structure.

The geotechnical design shall be done in a manner that is consistent with S6-00. Working Stress analysis is not required.

Seismic hazards with regards to slope instability, soil liquefaction, liquefaction induced ground movements and increase in lateral earth pressure shall be assessed. Soil-structure interaction, differential ground motion, and cyclic degradation effects shall also be considered in the design.

The effects of foundation and abutment stiffness and capacity, based on the best estimate of site conditions and soil parameters, shall be considered in analyzing overall bridge response and the relative distribution of earthquake effects to various bridge components.

7.2 SITE INVESTIGATION AND SEISMIC HAZARD ASSESSMENT

A report on potential seismic hazards at the bridge site is required. The potential seismic hazards shall be determined on the basis of field tests and laboratory testing as required.

7.2.1 Slope Instability

Initial analyses may be carried out using conventional pseudo-static methods to assess seismically induced slope instability for soil and rock slopes adjacent to the bridge. Such analyses shall be based on information obtained in the site investigation, including geometry of the slope, soil shear strength and other relevant geotechnical data at the north end. The slope instability assessments shall incorporate the effects of seismic forces transferred from the bridge superstructure where appropriate.

If the analyses show that slope instability is likely during or following the design earthquake, the effect of this instability on the bridge foundations, particularly regarding slope movement, shall be evaluated. If liquefaction is deemed to be the cause of the slope instability, analyses shall be carried out using appropriate reduced soil strength properties and increased pore water pressures for the potentially liquefiable zones unless ground improvement measures are provided.

If the movements are unacceptable, slope-stabilizing measures shall be taken to reduce such movements.

7.2.2 Soil Liquefaction and Liquefaction-Induced Ground Movements

An evaluation shall be made of the potential for liquefaction of foundation soils, and the impact of liquefaction on the bridge foundations and bridge superstructure. ATC-49:2003 (Guidelines for Seismic Design of Highway Bridges) provides procedures for this analysis.

If the analyses show unacceptable foundation movements, in terms of total movements and differential movements between adjacent foundations, one or more of the following measures shall be taken:

1) Use an appropriate foundation type, such as deep piles or piers that extend below the zones of liquefiable soils. These foundation elements shall be designed to withstand ground movement induced soil loads.

2) Employ soil improvement methods such as ground densification by vibro techniques, dynamic compaction, blasting, compaction grouting, or other suitable methods.

3) Design bridge structures to withstand the predicted ground movements.

7.2.3 Increases in Lateral Earth Pressure

Seismically induced increases in lateral earth pressure on the back of an abutment shall be included in design, where applicable.

7.2.4 Soil-Structure Interaction

The interaction of soil-foundation-structure system shall be evaluated where significant.

In global bridge modelling, equivalent linear or non-linear foundation soil springs shall be used. A range of possible soil spring stiffness shall be evaluated 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 variation in soil spring stiffness.

The participation of abutment foundations in the overall seismic response of the bridge shall be considered if this effect is deemed to be significant. The abutment participation shall reflect the structural configuration, the load-transfer mechanism from bridge to abutment system, the effective stiffness and force capacity of wall-soil system, and the level of expected abutment damage.

If soil liquefaction is deemed to be a potential problem or continuous soft clay layers are encountered within the embedment depth or directly below the foundations at the site, the following effects shall also be incorporated in the soil-structure interaction analysis, unless ground improvement measures are provided:

a) Soil strength reduction,

b) Cyclic degradation in foundation stiffness, and

c) Loading from earth pressures generated in sloping ground by lateral spread and settlement

7.3 RETAINING WALLS

AASHTO "Standard Specifications for Highway Bridges", 17th Edition, 2002 shall be the governing code for items other than structural design.

If limit equilibrium analysis can't meet the AASHTO requirements for design, dynamic analysis must be completed to prove the geotechnical design meets the performance criteria given in Section 2.3.

8 SEISMIC RETROFIT STRATEGY REPORT

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

- Additional project-specific seismic retrofit design criteria.
- A summary of design response spectra and ground motion time histories.
- Description of methodology and parameters for structural and geotechnical modelling and analysis.
- 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 must be included in the Drawings package. Summary logs must be in accordance with standard Ministry format as described in “Geotechnical and Materials Engineering Standards for Bridge Foundation Investigations (January 1991)”. Survey information on all logs will include local project referencing (station, offset and elevation above mean sea level) and UTM (NAD83) coordinates (Northing, Easting and UTM zone).