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SEISMIC DESIGN PROVISIONS - THE CANADIAN HIGHWAY BRIDGE DESIGN CODE

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Summary

This paper describes the new seismic design provisions of the Canadian Highway Bridge Design Code, due for publication by 1999. In particular the provisions which differ from those in the 1994 American Association of State Highway and Transportation Officials Load and Resistance Factor Code are highlighted.

INTRODUCTION

This paper describes the new seismic design provisions given in Section 4 of the Canadian Highway Bridge Design Code (CHBDC) [1], due for publication by 1999. In particular the provisions which differ from those in the 1994 American Association of State Highway and Transportation Officials Load and Resistance Factor Code (AASHTO LRFD) [2] are discussed.

The seismic zoning maps for the 1994 AASHTO LRFD Code were developed based on "firm-ground" horizontal accelerations having a probability of exceedance of 10% in 50 years. This is the same basis for the seismic zoning maps that appear in the 1995 National Building Code of Canada [3]. However the NBCC uses two parameters for each location, that is the acceleration and the velocity, each having a probability of exceedance of 10% in 50 years. While the Seismic Subcommittee of the CHBDC preferred the two-parameter approach, using both acceleration and velocity, it was felt that there were new design approaches on the horizon for North American codes. In particular the Uniform Hazard Spectrum (UHS) approach was being considered by the Canadian National Committee on Earthquake Engineering for possible inclusion in the next edition of the NBCC. The UHS approach offers the advantage of providing spectral values corresponding to two distinct periods for each location in the country. Because of the uncertainty of the format for the future NBCC it was decided to keep the basic AASHTO LRFD approach as an interim measure until the next code cycle rather than adopt the two parameter approach, which may change significantly in the future.

While the basic format of the AASHTO LRFD design code has been retained some significant changes were made in the development of the Seismic Design section, as described below.

IMPORTANCE CATEGORIES

New importance categories were chosen which differed from those used in the AASHTO LRFD code [2]. The new categories of "Lifeline bridges" and "Emergency-route bridges" replace the AASHTO designation of "Critical bridges" and "Essential Bridges". The definitions of the Importance Categories in the code are:

Lifeline Bridges - those that carry or cross over routes that must remain open to all traffic after the design earthquake (i.e., an event having a 10% probability of exceedance in 50 years or a return period of 475 years). Lifeline bridges must also be useable by emergency vehicles and for security/defense purposes immediately after a large earthquake (e.g., a 1000-year return period event).

Emergency-route Bridges - those that carry or cross over routes that should, as a minimum, be open to emergency vehicles and for security/defense purposes immediately after the design earthquake.

Additional guidance on the interpretation of these performance requirements for the Importance Categories is given in the Commentary to the code (see Table 1).

Table 1. Performance requirements

	Lifeline	Emergency-route	Other
Small to moderate earthquake	All traffic Immediate use	All traffic Immediate use	All traffic Immediate use
Design earthquake (475 year return period)	All traffic Immediate use	Emergency vehicles Immediate use	Repairable damage
Large earthquake (e.g.,	Emergency vehicles	Repairable damage	No collapse

1000 year return period)	Immediate Use		

ELASTIC SEISMIC RESPONSE COEFFICIENT

The same basic formulation for the elastic seismic response coefficient, C_{sm}, that is used in the AASHTO LRFD code [2] was used in the CHBDC [1], except an explicit Importance Factor was added to the formulation. The elastic seismic response coefficient, C_{sm}, for the m^{th} mode of vibration is:

$$Cost = [n_g C_g + C_c V_c + C_s (m_s + n_p (m_{sn} + m_{sp}) + m_p)] / WL \qquad [1]$$

where

T_m = period of vibration of the m^{th} mode

A = zonal acceleration ratio

S = site coefficient depending on the soil profile type, varying from 1.0 for Soil Profile Type I to 2.0 for Soil Profile Type IV.

I = Importance factor depending on the importance category, taken as:

- I = 3.0 for Lifeline bridges, but need not be taken greater than the value of R for the ductile substructure elements given in Table 2.
- I = 1.5 for Emergency-route bridges.
- I = 1.0 for other bridges.

For Soil Profile Type III or IV soils in areas where the Zonal Acceleration Ratio, $A \geq 0.3$, then C_{sm} need not exceed 2.0 AI.

For Soil Profile Type III or IV soils, C_{sm} for modes other than the fundamental mode which have periods less than 0.3 sec shall be taken as:

$$-\frac{P_i}{A} - \frac{P_i e}{S_b} + \frac{M_{dg}}{S_b} \geq -0.6 f'_{ci} \qquad [2]$$

For structures in which the period of vibration of any mode exceeds 4.0 seconds, the value of C_{sm} for that mode shall be taken as:

$$-\frac{P_e}{A} - \frac{P_e e}{S_b} + \frac{M_{dg} + M_{ds}}{S_b} + \frac{M_{da} + M_l}{S_{bc}} \leq 0.2\sqrt{f'_c} \qquad [3]$$

The variation of C_{sm} is shown in Fig. 1 for the case where the Importance Factor, I, is 1.0.

IMPORTANCE FACTORS AND FORCE MODIFICATION FACTORS

The AASHTO LRFD code [2] uses a Response Modification Factor for design which combines the so-called "structural ductility factor" with the Importance Factor. The AASHTO Response Modification Factor, which will be referred to as "R'" is used to divide the elastic design forces, but since it also includes an importance factor it can be thought of as R/I, where R is the Response Modification Factor to account for structural ductility and energy absorption and I is the Importance Factor. The AASHTO LRFD values of R' for different substructures are given in Table 2.

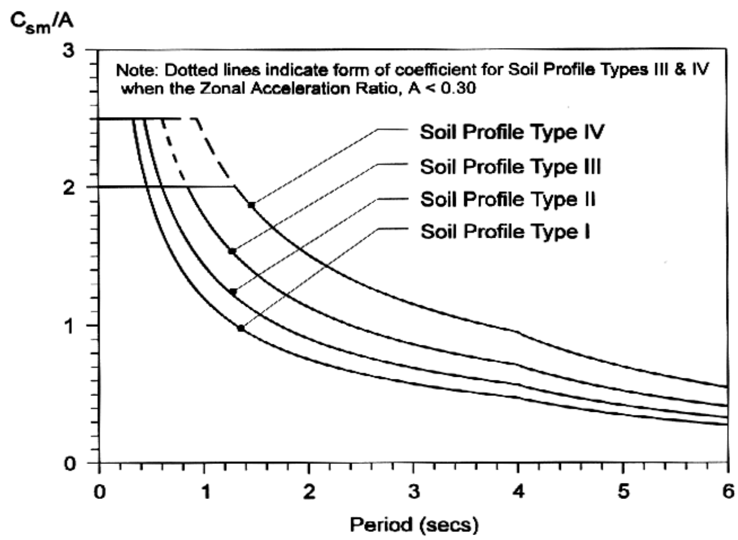

Figure 1. Normalized seismic response coefficients, Csm/A, for various soil profiles.

Table 2. Values of R' in the 1994 AASHTO LRFD Code

Substructure	Importance Category		
	Critical	Essential	Other
Wall-type piers - larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents			
- vertical piles only	1.5	2.0	3.0
- with batter piles	1.5	1.5	2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents			
- vertical piles only	1.5	3.5	5.0
- with batter piles	1.5	2.0	3.0
Multiple column bents	1.5	3.5	5.0

In the development of the seismic design provisions of the CHBDC [1] it was decided to clearly distinguish between the Importance Factor and the Response Modification Factor. This approach then parallels the approach taken in the National Building Code of Canada [3] where R reflects the capability of a structure to dissipate energy through inelastic behaviour. The categories of ductile substructure elements used in the CHBDC are identical to those cases specified in the AASHTO LRFD code except that a number of additional cases were added for ductile substructure steel elements. The values of R chosen for the different structural systems in the CHBDC correspond to the values of R' used in the AASHTO code for the case of "Other bridges", for which it was assumed that I equals 1.0 (see Table 3).

Table 3. Response modification factors, R, in the CHBDC

Ductile Substructure Element	Clause for Design and Detailing Requirements	R
Wall-type piers - larger dimension	4.7	2.0
Reinforced concrete pile bents		
- vertical piles only	4.7	3.0
- with batter piles	4.7	2.0
Single Columns		
- ductile reinforced concrete	4.7	3.0
-ductile steel	4.8	3.0
Steel or composite steel and concrete pile bents		
- vertical piles only	4.7 or 4.8	5.0
- with batter piles		3.0
Multiple column bents		
-ductile reinforced concrete	4.7	5.0
-ductile steel columns or frames	4.8	5.0
Braced frames		
-ductile steel braces	4.8	4.0
-nominally ductile steel braces	4.8	2.5

Note: The lateral load resisting substructure elements must be designed and detailed to be ductile, that is, having a minimum Response Modification Factor, R, of 2.0.

DESIGN AND DETAILING REQUIREMENTS

The design and detailing requirements in the CHBDC [1] are consistent with the general philosophy of capacity design, providing the ability for the ductile substructure elements to absorb significant amounts of energy, while designing other elements to be capacity-protected elements. The CHBDC clearly references the applicable clauses for the design and detailing requirements that must be satisfied if the R values in Table 3 are to be used. The values of R are also consistent with the general philosophy of capacity design and the R values used in the National Building Code of Canada [3].

Concrete Ductile Substructure Elements

The design and detailing requirements for concrete structures are based on the provisions in the 1994 AASHTO LRFD code, which are similar to the requirements for ductile elements in the 1994 CSA A23.3 Standard, "Design of Concrete Structures" [4].

Steel Ductile Substructure Elements

The CHBDC provides specific ductile detailing requirements for substructure elements consisting of ductile braced frames or ductile moment frames. These requirements are comparable to those currently enforced for ductile steel buildings in the National Building Code of Canada [3], with some minor modifications.

The objective is to ensure that steel substructure elements are detailed to be capable of exhibiting ductility consistent with the R-values assumed in their analysis and design. Experience in past earthquakes [7, 8, 9, 10] emphasizes the importance of ductile detailing in the critical elements of steel bridges. Research on the seismic behaviour of steel bridges [11, 12, 13, 14] and findings from recent seismic evaluation and rehabilitation projects [15, 16] further confirm that seismically induced damage is likely in steel bridges subjected to large earthquakes and that appropriate measures must be taken to ensure satisfactory seismic performance.

The same capacity design principles presented earlier also apply here. Explicit detailing requirements are presented in the CHBDC for ductile moment frames/bents and ductile concentrically braced frames used as substructure elements. The specifications refer to the Canadian Standard CAN/CSA S16.1 - 94, "Limit States Design of Steel Structures" [5] for information on ductile eccentrically braced frames if necessary, and recommends using an R factor of 5 in that case. Special bracing, energy-absorbing devices, or special ductile superstructure elements may also be used, but only if published research results, observed performance in past earthquakes, or special investigation can demonstrate their adequate performance, and if permitted by the Regulatory Authority.

A complete review of the fundamentals of ductile steel detailing is beyond the scope of this paper, and available elsewhere [e.g., 17, 18, 19]. However, a few noteworthy differences exist between the steel ductile detailing requirements featured in the CHBDC specifications and those commonly found in building design standards, and some of the most important nuances are summarized below.

Materials

The specifications require that ductile substructure elements be constructed of steels capable of developing a satisfactory hysteretic energy during earthquakes, even at low temperatures if such service conditions are expected. Typically, such steels having yield stresses, $F_y \leq 0.8F_u$, can develop a longitudinal elongation of 0.2 mm/mm in a 50 mm gauge length prior to failure, and have probable-to-nominal strength ratios consistent with those implied in these specifications.

Ductile moment frames and bents

The prevailing philosophy in the seismic resistant design of ductile frames in buildings is to force plastic hinging to occur in the beams rather than in columns, to better distribute hysteretic energy throughout all storeys and avoid soft-storey type failure mechanisms. However, for steel bridges such a constraint is not realistic, nor is it generally desirable. Steel bridges frequently have deep beams which are not typically Class 1 sections (i.e. compact sections as per U.S. designation), and which have larger flexural stiffness than their supporting steel columns. Moreover, bridge structures in Canada are generally "single-storey" structures, and all the hysteretic energy dissipated is concentrated in this "single storey". The CHBDC provisions are therefore written assuming that steel columns will be the ductile substructure elements in moment frames and bents. It is understood that extra care would be needed to ensure the satisfactory ductile response of multi-level steel frame bents since these are implicitly not addressed by these specifications.

For that reason, ductile detailing requirements are only specified for columns in ductile moment frames and bents. Hence, columns must be Class 1 sections (i.e. U.S. compact sections), must have lateral supports at the potential plastic hinge locations (near their top and base) and other lateral supports as necessary to limit the unsupported length to $980r_y / \sqrt{F_y}$. and cannot be subjected to factored axial compression in excess of $0.30A_gF_y$ due to the combined effect of seismic load and permanent loads if in seismic zones 3 and 4 (twice that value if in seismic zone 2). Other usual detailing requirements for ductile columns also apply [5].

Beams, panel zones, column bases, and moment resisting connections are designed as capacity protected elements, following the principles presented earlier, to remain elastic. To ensure the strong beam-weak column behaviour implied by the CHBDC, the sum of the factored resistance of the beams at any beam-to-column joint cannot be less than the sum of the probable resistance of the column(s) framing into the joint. The Probable Resistance of columns shall be taken as 1.25 times their nominal flexural capacity given by:

$$1.18 M_{px}\left[1 - \frac{C_f}{A_g F_y}\right] \leq M_{px} \qquad \textbf{[4]}$$

It is noteworthy that, during the Kobe earthquake, a number of steel box-columns supporting portions of elevated highways buckled, some rather severely, and at least two collapses occurred as a result of steel column failures. However, the large tubular steel piers used in Japan are uncommon in Canada, and the new CHBDC does not include any provisions for their design at this time.

Ductile concentrically braced frames

The same capacity design principles also apply to ductile braced frames. As normally done for ductile braced frames in building designs [5, 20]:

- Braces are the energy dissipating elements
- The load redistribution following the yielding or buckling of braces must be taken into account, and capacity-protected elements (e.g., columns, beams, beam-to-column connections and column splices) must be designed to resist the most detrimental condition that could result from this redistribution.
- Diagonal braces shall be oriented such that, in any planar frame, at least 30% of the horizontal shear carried by the bracing system shall be carried by tension braces and at least 30% shall be carried by compression braces.
- Chevron bracing, V-bracing, K-bracing and knee bracing are not considered as ductile concentrically braced frames.
- Braces must have a slenderness ratio, L/r, less than $1900/ \sqrt{F_y}$.
- Symmetrical open sections shall be Class 1 (i.e. compact sections). For other sections, the width-thickness ratios is limited to $145/ \sqrt{F_y}$. for angles, tees, and flanges of channels, $330/ \sqrt{F_y}$ for rectangular and square HSS, and $13\,000/ F_y$ for circular HSS.
- The factored compressive resistance of a brace must be reduced to account for the loss of compressive resistance under cyclic loading. This reduction is a function of the brace's slenderness ratio.

The CHBDC specifications also provide detailing requirements for concentrically braced frames with nominal ductility. Their energy dissipation capabilities are somewhat less than ductile braced frames, and this is reflected by a lower R-factor. However, Chevron-type braced frames are included in this designation.

IMPORTANCE FACTORS AND PERFORMANCE REQUIREMENTS

In selecting the Importance Factors, I, for the CHBDC, values were chosen which are similar to the importance factors implied in the R' factor given by AASHTO. The values of I were chosen to be 1.5 for Emergency-route bridges and 1.0 for Other bridges. For Lifeline bridges an importance factor of 3.0 was chosen, except that I need not exceed the value of R given in Table 3. It must be noted that substructure elements must be designed and detailed to have a minimum R value of 2.0. Figure 2 shows the variation of I with the different Importance Categories used in Section 4 of the CHBDC.

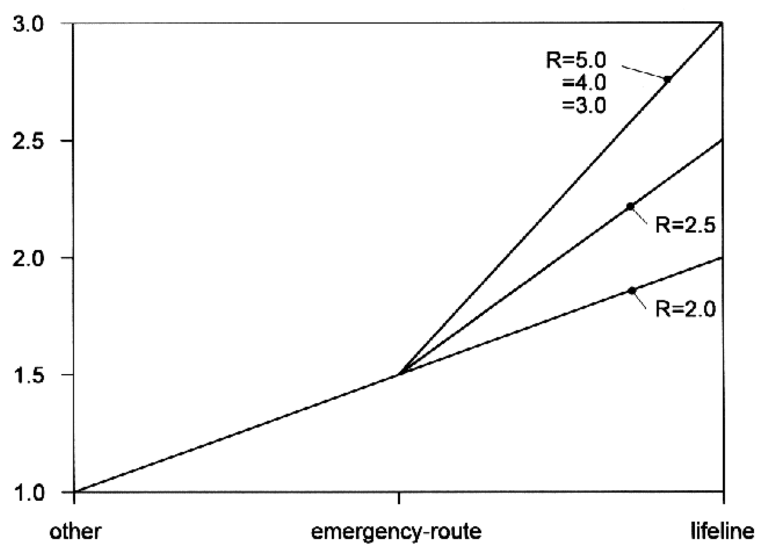

Figure 2 Variation of I with importance categpry and ductility of structural system

Figure 3 compares the value of R/I used in the CHBDC with the value of R' used in the AASHTO code. It is apparent from this figure that the CHBDC is slightly more conservative for the less ductile systems. This more conservative approach for cases with lower R values was adopted in order to provide further encouragement for designers to choose more ductile systems.

Values of I less than 3.0 for Lifeline bridges having R of 2.5 and R of 2.0 are justified if the performance criteria are examined. Figure 4 compares the expected ductility demand, μ_{demand}, for the cases of Lifeline bridges designed with R of 5.0, 4.0, 2.5 and 2.0. These ductility demands were determined at lateral force levels corresponding to an equivalent lateral design force, E_{design} (i.e., the design level earthquake). The structures designed with R of 5.0 and 4.0 would experience some inelasticity, while the structures designed with R of 2.5 and 2.0 would remain elastic. All of these cases are considered to meet the performance criteria for Lifeline bridges, that is that they must be capable of remaining open after experiencing the design level earthquake. The choice of an Importance Factor of 3.0 for the more ductile cases (i.e., where R is greater than 3.0) is necessary in order to limit the damage under the design earthquake. A constant Importance Factor of 3.0 for cases with R values below 3.0 would give rise to overly conservative designs which would exceed the performance requirements. For example, for the case of R of 2.0, if the value of I were 3.0, then the bridge would be designed to have a "yield strength" of 1.5 times the design level earthquake and the expected deformations under the design level earthquake would be well below the "yield strength". On the other hand, for the case of R of 2.0, limiting the Importance factor to R would mean that the structure would have a "yield strength" equal to the design earthquake level and hence would be on the verge of "yielding" under the design earthquake level (see Fig. 4(d)).

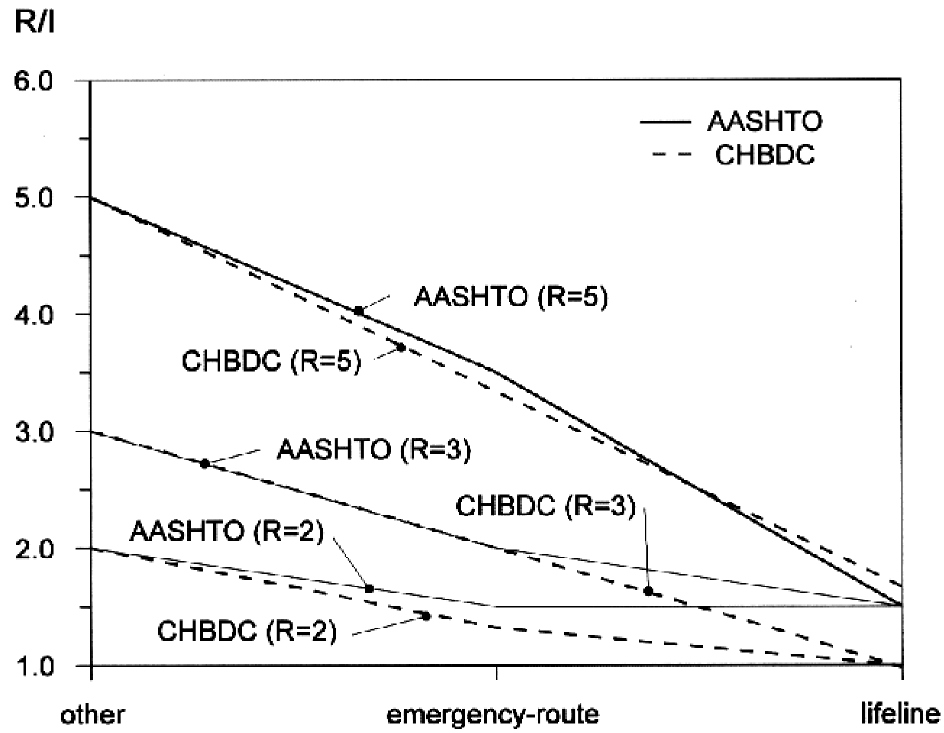

Figure 3– Comparison of R/I from CHBDC with corresponding parameter from AASHTO for different structural systems

SEISMIC BASE ISOLATION

The provisions for seismic base isolation are new requirements based on the 1997 AASHTO guidelines for base isolation [6] and include the design approach for isolation bearings together with performance specifications and testing procedures for isolation bearings.

SEISMIC EVALUATION OF EXISTING STRUCTURES

The CHBDC provides provisions for the evaluation of existing structures for Emergency-route bridges and Other Bridges, with Lifeline bridges requiring special studies. The minimum analysis requirements for seismic evaluation are summarized in Table 4. The Seismic Performance Zone is determined from the peak ground acceleration for 10% probability of exceedance in 50 years and from the Importance Category. The designation "LE" refers to limited evaluation and involves providing minimum seat widths or longitudinal restrainers and a minimum capacity for bearings. In addition, the potential for soil liquefaction, slope instability, approach fill settlements and increases in lateral earth pressures must be considered. The designation "SM" refers to single-mode elastic analysis and "MM" refers to the multi-mode spectral method. Push-over analysis and the time-history analysis methods are also permitted. Regular bridges are defined as having less than seven spans, no abrupt or unusual changes in weight, stiffness or geometry and no large changes in these parameters from span-to-span or support-to-support (excluding abutments).

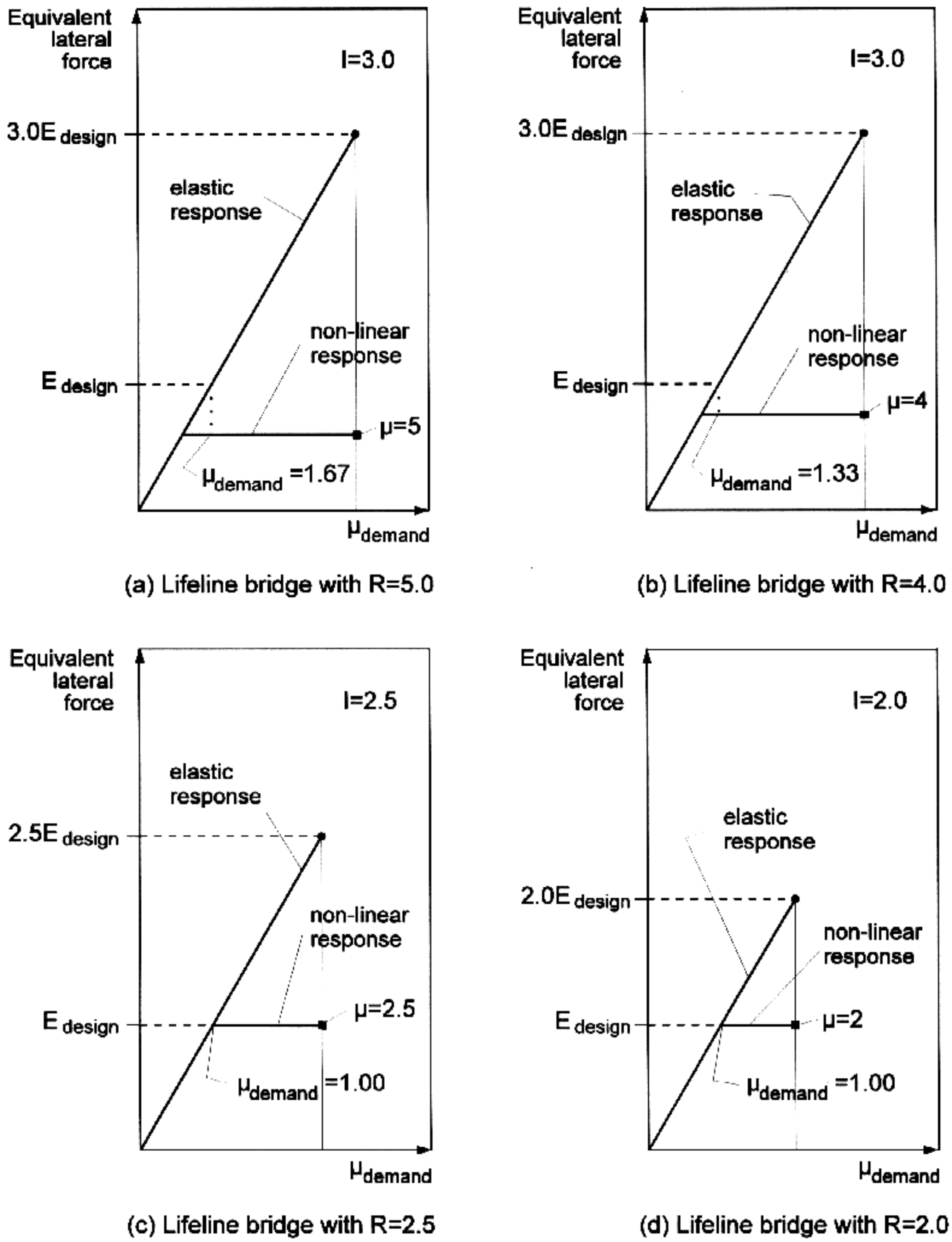

Figure 4 Influence of I and R on expected damage.

Table 4. Minimum analysis requirements for evaluation

Seismic Performance Zone	Single-span		Multi-span			
			Emergency-route		Other	
	Emergency-route	Other	Regular	Irregular	Regular	Irregular
1	None	None	None	None	None	None
2	None	None	LE	LE	None	None
3	LE	None	SM	MM	LE	LE
4	LE	LE	MM	MM	SM	MM

The CHBDC recognizes the important role that the regulatory authority plays in setting appropriate analysis and design requirements for evaluating existing bridges. Therefore, adjustments to the evaluation procedure are permitted if approved by the regulatory authority, such as the required analysis method, accounting for the remaining service life of the bridge and load cases to be considered. The load factors and load combinations are given as:

$$1.0D + 1.0E \qquad [5]$$

This represents a reduction from that required for new bridges where the earthquake effects are combined with minimum (0.8D) and maximum (1.25D) gravity loads. Combinations of orthogonal loading cases must be combined in the same manner as new bridges.

The evaluation procedure involves the calculation of the required response modification factor, R_{req}, from the following:

$$R_{req} = \frac{S_e}{C} \qquad [6]$$

where

S_e = seismic force effect assuming all members remain elastic, except as limited by capacities of other members

C = member reserve capacity after the effects of dead load have been considered.

Member capacities are calculated from the unfactored nominal resistances of the members. In the determination of the nominal resistances of members the code emphasizes the need to take account of the effects of all differences from the design and detailing requirements for new bridges including:

(a) influence of b/t ratios on local buckling in steel members,
(b) influence of slenderness ratios for steel members,
(c) influence of premature bond failures due to inadequate anchorage or splice length,
(d) influence of reduced concrete contribution to the shear resistance as the ductility demand increases in reinforced concrete members,
(e) influence of inadequately detailed beam-column and column-footing joints, and
(f) influence of defects or deterioration on member performance.

After determining S_e, C and R_{req}, the engineer must determine the appropriate response factor of the existing substructure, R_{prov}. The code requires that the determination of the overall performance and the R_{prov} must account for the following:
(a) the consequences of the specific detailing,
(b) consideration of all possible failure modes, and
(c) the expected length of inelastic deformations.
Results from reversed cyclic loading tests of structural components which are constructed to simulate the as-built details provide a means for determining a suitable R_{prov}.

Elements which have $R_{prov} \geq R_{req}$ are deemed acceptable, while those not meeting this requirement must undergo rehabilitation unless it can be demonstrated by non-linear analysis that the consequences would not be detrimental to the performance of the bridge.

SEISMIC REHABILITATION

The CHBDC provides guidance on the seismic rehabilitation of bridges indicating the following techniques:
(a) base isolation,
(b) increasing ductility without strengthening,
(c) addition of energy-dissipating devices,
(d) installation of restrainers,
(e) alteration of load paths,
(f) increasing support lengths,
(g) making provisions for inelastic hinging to occur,
(h) strengthening,
(i) improvement of liquefaction-prone soils, and
(j) stabilization of approach fills and adjacent slopes.

The CHBDC requires that the following design aspects be investigated when assessing seismic rehabilitation measures:
(a) increased stiffness due to strengthening must be accounted for,
(b) influence of rehabilitation on fatigue life must be assessed,
(c) influence of rehabilitation on alteration of load paths must be considered,
(d) influence of member strengthening on force demands on other members and joints must be assessed,
(e) rehabilitation measures should avoid damage to inaccessible foundations,
(f) if uplift occurs then guiding of the associated movement and prevention of support loss must be considered,
(g) if base isolation is used then consideration must be given to other loading cases (e.g., wind),
(h) the durability of the rehabilitation measures must be addressed,
(i) the restraint of thermal movement due to added restrainers must be considered,
(j) soil improvement may induce movements or tilting which must be addressed,
(k) the consequences of stage-wise rehabilitation must be considered,
(l) adequate inspection and maintenance of the rehabilitation must be addressed,

(m) a complete reanalysis of the rehabilitated structure must be carried out to assess performance,

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